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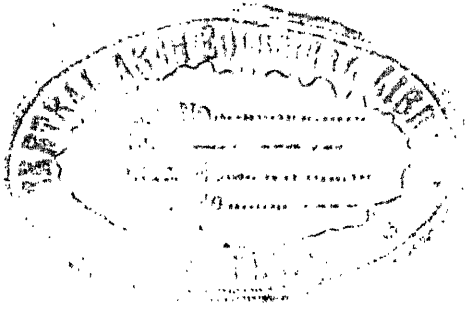
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A TEXT BOOK ON THE PRINCIPLES AND DETAILS OF MODERN CONSTRUCTION FOR THE USE OF STUDENTS AND PRACTICAL MEN

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BY

GEORGE A. MITCHELL,
F.R.I.B.A., M.I.STRUCT.E.,
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A. M. MITCHELL, B.Sc.(Eng.), A.C.G.I., M.I.STRUCT.E.,

Based on the work originally compiled by the late
CHARLES F. MITCHELL

PART 2.

ADVANCED COURSE

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PREFACE

TO THE THIRTEENTH EDITION

THE call for a new edition of this work has come in the midst of the most widespread and savage war of history, in which, in spite of the cessation of all private building, the study of building construction has actively continued in service quarters and in connection with various educational channels and institutions. My son and I have aimed at giving the text and illustrations a revision as thorough and careful as is practicable under war conditions. Opportunity has been taken to rectify some minor slips and misprints, and a number of chapters have been thoroughly corrected and rewritten to a greater or less extent, with additions where necessary; of this it is not necessary to give detailed particulars. Materials and methods which have gone out of use have been eliminated and replaced by fresh developments in construction, though it is naturally too early at this stage to forecast the undoubtedly extensive and far-reaching changes in building which will be introduced after the war, when the industry will unfailingly see a vast expansion and development. We have seen the effect of direct bombing, blast and incendiary fire on buildings old and modern, great and humble, and undoubtedly the study of this will have its effect on future methods of construction and use of materials.

The opportunity has been taken to incorporate the various changes in building bye-laws which have taken place in the last few years, and we must thank the public bodies concerned, who will be found mentioned in detail in the following Acknowledgment, for their kind co-

operation in permitting us to print their current versions. We are especially grateful to the Director of the British Standards Institution for allowing the inclusion of the new Specifications concerned with the Building Trade, of which a list follows at the end of this title-sheet. The Examination Questions have been replaced throughout by a selection from the most recent examinations available of the Royal Institute of British Architects, the Chartered Surveyors' Institution, the Institute of Builders, and the Institution of Structural Engineers, who must be thanked for their courtesy in permitting their inclusion.

We are glad that the Publishers have succeeded in obtaining a grant of extra paper from the Committee of the Publishers' Association that has enabled them to reissue a book of this great extent, which makes heavy demands, with its inevitably large edition, on available supplies during a period of necessarily severe paper restriction.

This book, with its fellow the "Elementary Course," celebrates this year the Jubilee of its first publication; fifty years is nearly two generations, and in the course of the half century something like 250,000 copies have been sold. We should like to express the hope that its usefulness to students of building construction may continue in the future as it has done in the past.

G. A. M.
A. M. M.

EALING.
Autumn, 1943.

PREFACE

TO THE TWELFTH EDITION.

As noted in the Preface to the Eleventh Edition, this work was thoroughly revised in 1929–30, but when the call came for a new edition I felt on careful consideration that there had been so many changes and developments in methods of construction that it was necessary for a further thorough overhaul, and this revision has occupied me during many months. It is unnecessary to recount in detail the various additions and changes that have been made, which will be apparent from an examination of the book, but I should like to mention that the chapter on Materials incorporates the result of recent research and the latest standard specifications with various tables; new material has been added to Foundations, Brickwork and Masonry and Graphic Statics, while Carpentry has been thoroughly overhauled, with the addition of much new material on Tubular Scaffoldings, Cranes, Formwork, etc., and Pillars has been brought into line with the Code of Practice and with the addition of new calculating examples; this has also occurred in the case of Riveting and Girders. The chapters on Ferro-Concrete and Fire-Resisting Construction have been completely rewritten and re-arranged; those on Roofs, Joinery, Staircases, Hot Water Heating and Sanitation have also been revised, with a number of new examples. I have endeavoured to incorporate the various new Byelaws and regulations which it is advisable to include, and have also included the current British Standard Specifications affecting the subjects dealt with in

this work. The Examination Questions have been entirely replaced by those set at the latest examinations of the Royal Institute of British Architects, the Chartered Surveyors' Institution, the Institute of Builders, and the Institution of Structural Engineers.

The work has been completely reset in new type, many fresh illustrations added and a large number of old ones redrawn, while I have sought to keep the book within manageable compass by eliminating certain sections which no longer form a part of current practice.

I can only hope that the work will continue its usefulness as a text book for students and practical workers.

GEORGE A. MITCHELL.

EALING, W. 5.
October, 1936.

ACKNOWLEDGMENT.

I MUST express my obligation to the following, who have generously granted me permission to reprint the whole or portions of various reports, byelaws, specifications, notes, examination papers, etc., together with the accompanying illustrations:—

The Director of the British Standards Institution, 28 Victoria Street, London, S.W.1. A list of the Specifications dealing with the Building Trades will be found on pages xiii–xvi.

The Controller of His Majesty's Stationery Office for the "Report of the Reinforced Concrete Structures Committee of the Building Research Board with Recommendations for a Code of Practice for the Use of Reinforced Concrete in Buildings," pp. 667–724.

The Clerk to the London County Council for "By-laws made by the London County Council in pursuance of the London Building Act (Amendment), Act, 1935, Part IV, Walls and Piers, Section 1 (39–50), Section 2 (51–57), Section 3 (58–62); Paragraphs 124–131, Tall Chimneys; Paragraph 143 (3–4), Shop Fronts; and Section 24 of the London County Council (General Powers) Act, 1928, with respect to water-closets, privies, and cesspools, and the proper accessories thereof in connection with buildings." Sections 1–7 with footnotes: "Drainage By-laws made by the London County Council in March 6th, 1934, regulating the dimensions, form and mode of construction, and the keeping, cleansing and repairing of the pipes, drains, and other means of communicating with sewers

and the traps and apparatus connected herewith." No. 3065, forming, with footnotes, pp. 927–953.

The Ministry of Health for the Public Health Act, 1936, Part II, 81 (1 and 2), p. 890.

The Councils of The Royal Institute of British Architects, The Institute of Builders, The Chartered Surveyors' Institution, The Institution of Structural Engineers, for the Examination Questions on pp. 997 *et seq.*

Messrs. Frederick Braby & Co. Ltd., of Glasgow, for "Metal Staircases," pp. 657–660.

The Expanded Metal Co. Ltd., of West Hartlepool, for "'Ribmet' Expanding Metal Lathing," pp. 49–51.

Messrs. Henry Hope & Sons Ltd., of Smethwick, Birmingham, for "Substitutes for Joinery Work," pp. 857–868.

Steel Scaffolding Co. Ltd., of Regent Street, London, for "Steel Tubular Scaffolding and Construction," pp. 396–407.

Trussed Steel Concrete Co. Ltd., of Manchester, for "'Hy-Rib' Metal Laths," pp. 51–53.

GEORGE A. MITCHELL.

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BUILDING.

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B.S. No.

STRUCTURAL MATERIALS

(A) IRON AND STEEL

4–1932. Channels and Beams for Structural Purposes, Dimensions and Properties of [*Add.* April, 1934]. [Partly superseding No. 6–1924.]
4a–1934. Equal Angles, Unequal Angles and Tee Bars for Structural Purposes, Dimensions and Properties of [Partly superseding No. 6–1924].
6–1924 (Extract from). Bulb Angles, and Bulb Plates for Structural Purposes, Dimensions and Properties of [See Nos. 4 and 4a]. (1/–, 1/3.)
[N.B.—Approximate Formulæ for all Sections are included in the Extract from No. 6–1924.]
15–1936. Steel for Bridges, etc., and General Building Construction [*Add.* Feb., 1938 and Feb., 1941].
405–1930. Expanded Metal (Steel).
548–1934. High Tensile Structural Steel for Bridges, etc., and General Building Construction [*Add.* May, 1936].
785–1938. Rolled Steel Bars and Hard Drawn Steel Wire for Concrete Reinforcement.
†968–1941. High Tensile (Fusion Welding Quality) Structural Steel for Bridges, etc., and General Building Construction.

(B) CEMENT; LIME AND AGGREGATES

12–1940. Ordinary Portland and Rapid-Hardening Portland Cement [*Add.* March, 1942].
146–1941. Portland Blastfurnace Cement. Not exceeding 65% Blastfurnace Slag [*Add.* March, 1942].
877–1939. Foamed Blastfurnace Slag for Concrete Aggregate.
882–1940. Natural Aggregates up to 1½ in. Nominal Maximum Size for Concrete Structural Purposes including Roads.
890–1940. Building Lines. (3/6, 3/9.)
915–1940. High Alumina Cement.
†1047–1942. Air-Cooled Blastfurnace Slag Coarse Aggregates.

(C) TIMBER

*531–1934. Grading for Birch Plywood (Suspended).
588–1935. Grading of Plywood (Veneered with Oak, Mahogany, Walnut, Teak and other Ornamental Woods).
940–1941. Grading Rules for Structural Timber. (The 800 lb. F. Grade Redwood, Scots Pine, European Larch, Douglas Fir (Home grown).)

STRUCTURAL UNITS

(A) BRICKS AND BLOCKS

187–1942. Sand Lime (Calcium-Silicate) Bricks.
492–1933. Precast Concrete Partition.
†657–1941. Common Building Bricks, Dimensions of. (1/–, 1/3.)
728–1937. Precast [illegible] Slabs.
834–1939. Precast Concrete [illegible] Walls.

B.S.
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(B) DAMP PROOF COURSES

743–1941. Materials for Horizontal Damp Proof Courses, including Classification for Bituminous Damp Proof Courses.

(C) PIPES AND TUBES, HEATING, VENTILATING, WATER AND SEWAGE

40–1908. Cast Iron Low Pressure Heating Pipes.
41–1908. Cast Iron Flue or Smoke Pipes.
65–1937. Salt-Glazed Ware Pipes (including Taper Pipes, Bends and Junctions).
78–1938. Cast Iron Pipes (Vertically Cast) for Water, Gas and Sewage and Specia Casting. [illegible Nov., 1938]. (5/–, 5/5.)
99–1922. [illegible] *Id.* Oct., 1927). (3/6, 3/9).
143–1938. [illegible] Copper Alloy Pipe Fittings (Screwed B.S.P. Taper Thread) for Steam, Water, Gas and Oil. (3/6, 3/9.)
416–1935. Cast Iron Spigot and Socket Soil, Waste, Ventilating and Heavy Rainwater Pipes.
437–1933. Cast Iron Spigot and Socket Drain Pipes.
460–1932. Cast Iron Spigot and Socket Light Rainwater Pipes (Cylindrical).
486–1933. Asbestos Cement Pressure Pipes.
497–1933. Cast Iron Manhole Covers and Frames (Light).
*504–1933. Drawn Lead Traps (Under Revision).
539–1937. Salt-Glazed Ware and Salt-Glazed Enamelled Fireclay Drain Fittings, Dimensions of.
540–1937. Salt-Glazed Glass (Vitreous) Enamelled Fireclay Pipes (including Taper Pipes, Bends and Junctions).
567–1934. Asbestos Cement Spigot and Socket Flue Pipes and Fittings of Gas Fired Appliances, Dimensions and Workmanship of.
569–1934. Asbestos Cement Spigot and Socket Rainwater Pipes, Gutters and Fittings, Dimensions and Workmanship of [*Add.* Nov., 1939].
582–1934. Asbestos Cement Spigot and Socket Soil, Waste and Ventilating Pipes and [illegible] Workmanship of.
602–[illegible] Chemical Purposes [*Add.* June, 1941, and March, 1942].
603–1941. Lead Pipes [B.N.F. Ternary Alloy (No. 2).]
659–1936. Light Gauge Copper Tubes [*Add.* June, 1938].
674–1936. Rubber Joint Rings for Water Mains and Sewers.
715–1936. Sheet Metal Cylindrical Flue Pipes and Fittings and Accessories for Gas Fired Appliances.
788–1938. [illegible] Tubulars, Gas (Light), Water (Medium), Steam (Heavy) [illegible] 938 and Jan., 1939].
789–1938. [illegible] Gas (Light), Water (Medium), Steam (Heavy) Qualities (*Add.* March, 1938 and Jan., 1939].
†789a–1940. Steel Tubes and Tubulars, Light Weight and Heavy Weight Qualities (Revised Weights.)
835–1939. Asbestos Cement Flue Pipes and Fittings (Heavy Quality) for Domestic Heating Stoves [*Add.* June, 1941].
864–1939. Capillary Joints for Copper Tubes (Internal Dimensions of Sockets).
945–1941. Rubber and Insertion Jointing for Flange and Similar Joints subject to Water Pressure.

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402–1930. [illegible] 1937].
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550–1934. [illegible]
680–1936. Welsh Roofing Slates.
690–1940. Asbestos-Cement Slates and Unreinforced Flat Sheets and Corrugated Sheets.
747–1937. Bituminous Roofing Felts.
849–1939. Plain Sheet Zinc Roofing, Code of Practice for [*Add.* June, 1939].
988–1941. Mastic Asphalt for Roofing, Type A (Limestone Aggregate).
†989–1941. Bituminous Roofing Felt (including Classification and method of laying).

(E) JOINERY

†459–1941. Doors (Framed, Flush, Ledged, Braced and Matched) for Internal and External Purposes [*Add.* June, 1941].
583–1934. Wooden Gates.
584–1934. Wood Mouldings (Architraves, Picture Rails, Dado Rails and Skirtings).
585–1934 Wooden Stairs.

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644–1935. Wooden Windows (Casements and Solid Frames, Double Hung Sashes and Cased Framed Windows).
645–1935. Cupboard Fronts and Dressers.
1018–Part 1.—1942. Timber in Building Construction. Floors.

F) GLASS AND GLAZING
952–1941. Glass for Glazing, including Definitions and Terminology of Work on Glass. (3/6, 3/9.)
973–1941. Code of Practice for the Glazing and Fixing of Glass for Buildings.

MISCELLANEOUS UNITS

†417–1941. Galvanised Mild Steel Cisterns, Tanks and Cylinders.
455–1932. Steel Cased Mortice Locks (5-in. and 6-in.), Dimensions of.
493–1933. Cast Iron Airbricks and Gratings (for use in Brickwork).
699–1936. Copper Cylinders for Domestic Purposes (Grades 1, 2 and 3).
758–1937. Domestic Hot Water Supply Boilers burning Solid Fuel—
758—Part 1.–1937. Specification.
758—Part 2.–1937. Method of Testing.
774–1938. Under-Floor Steel Ducts for Electrical Services, with Fittings.
779–1938. Cast Iron Boilers for Central Heating and Hot Water Supply [*Add.* Nov., 1939].
780–1938. Riveted Steel Boilers for Hot Water Central Heating and Hot Water Supply [*Add.* Nov., 1939].
815–1938. Under-Floor Non-Metallic Ducts for Electrical Services, with Fittings.
†990–1942. Metal Windows and Doors [*Add.* May, 1942].
†1010–1942. Bib, Pillar, Globe and Stop Taps from ¼ in. to 2 in. size and Ball Taps.

CODES OF PRACTICE

308–1927. Engineering Drawing Office Practice.
449–1937. Use of Structural Steel in Building [*Add.* May, 1940].
538–1940. Metal Arc Welding in Mild Steel as applied to General Building Construction [*Add.* August, 1940].
617–1935. Identification of Pipes, Conduits, Ducts and Cables in Buildings [*Add.* July, 1937].
648–1935. Unit Weights of Building Materials, Schedule of.
693–1940. Oxy-Acetylene Welding in Mild Steel.
849–1939. Plain Sheet Zinc Roofing, Code of Practice for [*Add.* June, 1939].
913–1940. Pressure Creosoting of Timber.
938–1941. Metal Arc Welding as applied to Tubular Steel Structural Members.
973–1941. Code of Practice for the Glazing and Fixing of Glass for Buildings.
1043–1942. Code of Practice for the Provision of Engineering and Utility Services in Buildings.

DEFINITIONS, NOMENCLATURE AND METHODS OF TESTING

373–1938. Testing Small Clear Specimens of Timber, Methods of.
476–1932. Fire-Resistance, Incombustibility and Non-Inflammability of Building Materials and Structures, Definitions for.
499–1933. [illegible] for.
560–1934. [illegible] 6, 3/10.)
565–1938. [illegible]
589–1935. [illegible] Sources of Supply).
661–1936. Glossary of Accoustal Terms and Definitions. (3/6, 3/9.)
685–1937. Trade Headings and Specification Items for Building Work, Sequence of.
812–1938. Sampling and Testing of Mineral Aggregates, Sand and Fillers. Methods for the. (3/6, 3/9.)
874–1939. Definitions of Heat Insulating Terms and Method of Determining Thermal Conductivity and Solar Reflectivity.
881–1939. Nomenclature of Hardwoods (including Botanical Species and Sources o Supply). (3/6, 3/9.)
892–1940. Highway Engineering Terms, Glossary of. (5/–, 5/4.)

CRANES AND LIFTING GEAR

327—Part 1.—1934. Derrick Cranes (Power Driven) [*Add.* May, 1934, and March 1942].

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327—Part 2.—1933. Derrick Cranes (Hand Operated) [*Add* March, 1942].
329–1939. Round Strand Steel Wire Ropes for Lifts and Hoists.
357–1930. [illegible]es (Contractor's Type) [*Add.* Dec., 1941].
394– [illegible] Crane Chain [*Add.* Feb., 1937].
781– [illegible] and Rings, Links Alternative to Rings, Egg [illegible] July, 1938]. (3/6, 3/9.)
825–1939. Mild Steel Shackles for Lifting Purposes.

UNCLASSIFIED

138–1935. Portable Chemical Fire Extinguishers of the Acid Alkali Type (excluding Foam Type).
144–1936. Coal Tar Creosote for the Preservation of Timber (Types A, A2 and B [*Add.* Feb., 1941].
219–1932. Soft Solders (Grades A, B, C, D, E, F, G, H, J and K).
336–1936. Fire Hose Couplings (including Screwed Outlets for Hydrants), Suction [illegible] Branch Pipes and Nozzle Connections.
441– [illegible] Rosin Filled.
481– [illegible] and Perforated Plate Sieves and Screens for Industrial Purposes.
544–1934. Linseed Oil Putty, Types 1 and 2 [*Add.* Feb., 1939 and Dec., 1940].
606–1935. [illegible]
745–1937. [illegible] or Powder, Jelly or Liquid, and Casein Glue) [*Add.* Oct., 1937].
776–1938. Materials for use in the Manufacture of Magnesium Oxychloride Flooring Compositions.
792–1938. Mild Steel Dust-Bins [*Add.* Aug., 1939 and Aug., 1940
810–1938. Plain Linoleum and Cork Carpet.
876–1939. Hand Hammers [*Add.* April 1940].

"*Add.*" signifies that an Amendment is issued with this Standard.
† War Emergency Standard.

Prices: 2/– net, post free 2/3 per copy, unless otherwise stated.

Additional specifications for British Standard Building Standards are included in the B.S./A.R.P. Series (separate list of these is available). A complete list of British Standards will also be supplied on application.

BRITISH STANDARDS INSTITUTION, PUBLICATIONS DEPARTMENT, 28, VICTORIA STREET LONDON, S.W.1.

VOL. II

ADVANCED BUILDING CONSTRUCTION

CHAPTER I

LIMES AND CEMENTS

Limestones.—Stones consisting entirely or in a great part of carbonate of lime, $CaCO_3$, are known as limestones, and are quarried chiefly in the form of chalk, which is a soft white rock of almost pure calcium carbonate, limestones, and marbles. For descriptions of the latter two, see Chapter on Stones.

The resultant of limestones after burning is lime, which is used in building operations (*a*) as a matrix for concrete, (*b*) in the preparation of mortar for bedding bricks or stones in walling, and (*c*) as a cementing material in the plaster used for the covering of walls.

Classification.—Limes are classified under the following heads :—

1. Pure or rich limes.
2. Poor limes.
3. Hydraulic { Feebly hydraulic. / Moderately hydraulic. / Eminently hydraulic.

Limestones are never found absolutely pure in Nature, but are either mixed with impurities or in combination with them, which impurities, if soluble in acids, are useful in

accelerating the setting action, but if insoluble in acids are valueless for that purpose.

British Standard Specification No. 890 gives particulars of the detailed requirements for building limes.

Rich Limes.—Limes are said to be rich or pure when the impurities insoluble in acids do not exceed 3 per cent. by weight of the whole.

For plastering, rich, pure, or fat limes only should be used, because of their readiness to slake, and their consequent non-liability to blister as compared with hydraulic limes.

Poor Limes are those containing from 15 to 30 per cent. of impurities insoluble in acids. They possess the general properties of rich limes, but in a less degree. They take longer to slake, and do not increase in bulk to such an extent as the rich limes. They do not take such a large ratio of sand, owing to the foreign matter they already contain.

Hydraulic Limes contain a quantity of combinable substances other than lime, such as silica and alumina, which on being burnt form calcium aluminate and calcium silicate, together with a portion of lime, the measure of these bodies up to a certain point being the measure of the hydraulicity. These bodies render the limes independent of external agents for their setting properties.

Limes containing 6 to 16 per cent. of these useful substances are termed feebly hydraulic, those containing 16 to 26 per cent. moderately hydraulic, and those from 26 to 36 per cent. eminently hydraulic.

Hydraulic limes were at one time used as the matrix for lime concrete. They were, in fact, a form of natural Portland cement with similar setting properties ; but their use, except possibly under purely local conditions, has now been superseded by Portland cement.

Typical Analyses.—The following are typical analyses of some of the limes in general use in London and in the south of England :—

	Chalk Lime (White).	Grey Stone (Yellow).	Grey Stone, Medway.	Blue Lias Lime, Rugby.
Lime (CaO)...	91·22	80·24	77·86	61·17
Silica (SiO_2)...	1·60	11·40	13·26	20·04
Alumina (Al_2O_3) ...	0·80	4·60	6·4	4·97
Ferric Oxide (Fe_2O_3)				6·10
Magnesia (MgO)	1·50	0·50	0·57	0·95
Carbon-dioxide (CO_2) and water (H_2O)...	4·03	2·01	1·57	4·03
Alkalies (K_2O and Na_2O) ...	0·85	1·25	0·00	1·08
Sulphuric Anhydride (SO_3)	—	—	0·34	1·09
Insoluble residue	—	—	0·00	0·57
	100·00	100·00	100·00	100·00
Authority	J. L. Peters, Merstham Lime Works.	J. L. Peters, Merstham Lime Works.	Associated Portland Cement Manufacturers.	Nelson & Co., Rugby.

From the time limestone is quarried to the setting of the lime in the work, four processes are gone through, viz., burning, slaking, mixing with sand, and setting.

$CaCO_3$ is a substance insoluble in pure water, and is unable to combine with carbonic acid gas when in this form, but is rendered suitable by calcination, which drives off the carbon dioxide (CO_2), leaving calcium oxide (CaO) together with any impurities contained in the stone.

Burning of Limestones.—The system of calcination may be continuous or intermittent.

In the continuous system the lime is gradually removed from the bottom of the kiln and fresh lime and fuel added on the top. This is an economical method, but the resultant lime is unequally burnt.

The intermittent method consists in firing and burning a kiln full of limestone each time. After cooling the kiln is emptied of the burnt lime. The operation is then repeated. A more evenly burnt lime is produced by this method. Each complete operation takes about one week.

The kilns are classed as tunnel or flare kilns, and each may be worked on the continuous or intermittent systems. Usually the tunnel kilns have alternate layers of coal and limestone, whilst in the flare kilns the coal is lowermost and the limestone is packed above.

For the burning of pure dense limestones about $\frac{1}{4}$ of their weight of coal, and for hydraulic limestones about $\frac{1}{6}$ of their weight of coal is necessary in each case.

Limestones are burnt in kilns of various shapes, the most common being a cylindrical brick or stone casing lined with fire-brick, having a draw-hole at one part to apply the fuel; this arrangement is termed a flare kiln. A rough dome is built with the limestones, forming a chamber to contain the fire; the kiln is then filled about the dome, the top of the kiln being usually covered by a shed to protect it from the weather.

In burning the limestones, the heat should be applied gradually, otherwise the separation of the carbon dioxide from the stones takes place with such rapidity that they crumble to pieces.

It is imperative that the calcined stones should be withdrawn from the kiln as soon as the carbon dioxide has been driven off, this being determined as follows: While any carbon dioxide remains in the stones they will be of a dark red colour, but when this has passed off the colour changes to a brilliant white glow; at this point the stones should be withdrawn, or they will be overburnt, the result being a lime which is very difficult to slake, this action not taking place for a considerable time after the mortar or plaster has been made, rendering the lime unreliable for use, as it would commence to blow when bedded in the work. A somewhat similar result occurs with underburnt limes.

Slaking.—The object of slaking lime is to form a calcium hydrate, thus rendering it quickly in a fit condition to readily combine with the CO_2 to form crystals of calcium carbonate, the formation of the latter being a necessary condition for strength in a mortar. Slaking is induced by adding water to quicklime; these on combining give off great heat and form steam, causing the lime to expand, burst, and disintegrate with a series of small explosions,

forming a calcium hydrate, $Ca(OH)_2$, a white powder. The slaked lime thus formed is soluble, and hence when more water is added some of this dissolves and forms a saturated solution.

The soluble lime in a saturated solution is ready for the absorption of CO_2, which always exists in the atmosphere.

If quicklime be left exposed to the air, it absorbs CO_2, which under these conditions renders it inert, as the resulting carbonate is not crystalline.

Slaking of the lime is an important process in the manufacture of mortar, and it is imperative that every particle of quicklime must be thoroughly slaked, for if any unslaked portions are built in the work it will by its subsequent expansion disturb the rest of the work.

To obviate this failing, the mortar after mixing should always be left to temper, covered over sometimes with a layer of sand, for at least a week to one month before being used.

The purer the lime is the longer it may safely be matured.

Hydralime.—With limes even from the same quarry there is always a considerable amount of variation in composition, therefore the result must always be of a speculative nature. Overburnt and underburnt particles both render the time for slaking variable. Added impurities from the fuel, etc., all affect the final result. Great care must be taken in the slaking, and time must be allowed for slow slaking particles to become properly hydrated, if pitting and blowing is to be eliminated. Improved methods are now used in the preparation of lime. Special kilns are employed in which producer gas is employed for the burning. Added impurities are thus avoided. The temperature is controlled and the final product is uniform in character. After this, water is added in the right amount to correctly hydrate every particle, and the material leaves the works as a fine powder in which form it is known as "hydralime." In this form it has many advantages to the builder, both for construction and for plastering. (1) It can be mixed with the sand more intimately before water

is added; (2) it can be mixed as needed and used direct; (3) it makes a stronger mortar and hardens quicker than ordinary lump lime; (4) it can be stored and kept for a considerable time without deterioration, and being already slaked, the risk of fire is eliminated. As there is no swelling as with lump lime, it does not burst the sacks. The first cost of hydralime is about 20 per cent. more than that of lump lime, but this is offset by the greater proportion of sand that can be used in the mixer. The strength of hydralime mortar and the rapidity of setting can be considerably increased by the addition of a proportion of Portland cement ranging from 5 to 25 per cent. by volume. Lime so mixed must be used within a few hours of gauging. Cement mortar for constructional work made with sharp sand is apt to work short. Such mortar can be made to work fatter by the addition of from 10 to 15 per cent. of hydralime.

Sand is a form of silica (SiO_2); it is added to lime in the preparation of mortar, (1) to counteract the excessive shrinkage that takes place with pure lime mortar, (2) to assist in the hardening by forming ducts through which the necessary CO_2 can have access and act upon the particles behind the surface, and (3) to increase the bulk of the mass (sand being much cheaper than lime).

The sand for mortar should be free from all earthy or clayey matter; it should have sharp angles and a rough surface.

The best sand for building purposes is that known as pit sand. The next in order is river sand, which is obtained from the banks and beds of rivers; this kind is not considered so good as pit sand, the grains being rounded and worn smooth, the adhesive value being thus reduced. River sand is, however, largely used for plasterers' work, it being fine and of a light colour. The grains in sea sand are similar to river sand, round and smooth. Sea sand should never be used for plastering or other building work, as it effloresces, thereby causing a wall to be damp for a considerable time.

Sand should be fine for plasterers' work, and moderately coarse for bricklayers' work; it is usually screened and

sometimes sifted to remove any large stones or shingle that it may contain.

Loamy or dirty sand should be washed before being used. This is usually effected by placing the sand in a vessel through which a stream of water is constantly passing, the sand at the same time being agitated to separate it from the foreign matter, which latter becomes suspended in the water and passes off.

Sand for all coats of plasterers' work is better washed, although for the first coats this is often neglected.

Setting of Lime.—The setting of lime depends on the absorption of CO_2 from the atmosphere by the particles of slaked lime in solution in the mortar, the carbon dioxide being soluble in water. The $Ca(OH)_2$ with excess of CO_2 combine to form $Ca(HCO_3)_2$ which decomposes on evaporation into crystals of $CaCO_3$, the H_2O helping to dissolve the next particle, forming it into a saturated solution, and putting it into a condition to take up a molecule of CO_2; this in its turn repeats the action already described, and crystals of $CaCO_3$ are formed. The crystals always have a tendency to adhere to something rough and hard, such as sandy particles or the surfaces of bricks; for this reason the addition of sand up to a certain ratio increases the strength of the mixture, the best ratio being one part lime to one of sand, the maximum being one of pure lime to three parts of sand.

A long time elapses before pure limes harden, owing to their depending upon external aid to attain this state. If lime alone is used the surface sets and forms an impervious layer, and so checks the CO_2 from acting on those particles below the surface, the moisture in which evaporates and leaves it in the state of a powder; even when a large proportion of sand is used and the mass made porous, the supply of CO_2 must necessarily be small, and a long time elapses before the material hardens. Pure lime mortar built in thick walls never hardens nor sets, but crumbles into a friable powder.

For this reason pure limes should be avoided for constructional work, and cement, which does not depend on external aid to set, be used.

Setting of Cement (and Hydraulic Limes).—The setting of cement does not depend upon the absorption of carbon dioxide from the atmosphere. It is an independent chemical reaction involving the formation, and hardening in combination, of hydrated calcium silicate and hydrated calcium aluminate. A typical Portland cement would be represented somewhat as follows :—

$$(CaO)_2 . Al_2O_3 H_2O + 3 (2 CaO, SiO_2 H_2O)$$

In hydraulic limes a certain amount of carbon dioxide (CO_2) should be absorbed, there being in most of these a small percentage of free lime.

As these limes and cements do not depend on external agencies for their setting properties, they are able to set in the centre of thick walls and under water. This renders them valuable for all constructional work.

Hydraulic limes possess none of the marked characteristics of the pure limes in slaking : they do not increase very much in bulk, and the slaking takes a much longer time to accomplish.

Blue lias lime, from the rocks of the lias geological formation, is one of the best natural hydraulic limes. It is obtainable from Lyme Regis in Dorset, Keynsham in Somerset, Shipston and Rugby in Warwickshire, Barrow-on-Soar in Leicestershire, Aberthaw in Glamorganshire, also in Flintshire, Lincolnshire, and Yorkshire.

Hydraulic Lime Mortar.—The strong hydraulic limes are usually ground into powder to facilitate the slaking. Slake the lime by sprinkling it lightly with water, then turn it up together in a heap, and cover it with sand. After 24 hours it may be made into mortar by adding the proper proportions of sand and water.

One part of lime and 2 parts of sand make excellent mortar.

Gypsum—Chemical Analysis.—Gypsum is a native hydrated sulphate of lime, occurring as a soft stone ($CaSO_4.2H_2O$) usually of a more or less crystalline texture, and varying in colour from white to shades of brown and grey to black. It is found in Derbyshire, Nottinghamshire,

Cheshire, Westmorland, and in great abundance near Paris. It contains in every 100 parts by weight 46·5 parts of SO_3, 32·8 parts of CaO, and 20·7 parts of H_2O. The very fine-grained pure white variety of gypsum is termed alabaster; the transparent, selenite.

Plaster of Paris.—Plaster of Paris is produced by the careful heating of gypsum in iron vessels to about 280° Fahr. During the process the gypsum loses three-fourths of its water, and thus the formula for plaster of Paris will be $2CaSO_4.H_2O$. It is ground to powder. If water is added it takes up as much as it lost in the calcination process and sets very rapidly.

Characteristics.—This is the only building material which at the time of setting expands in volume, this property making it valuable for filling up holes and other defects.

Uses.—It is used for making ornaments for ceilings, etc., and where plentiful may be used in all parts of a building where it is not exposed to the weather, for which it is unfit, being very soluble in water.

Its effect when mixed with the ordinary limes is to arrest the slaking and to increase the rapidity of setting.

In the manufacture of Portland cement a small quantity of gypsum is sometimes added to improve the soundness and produce a slow-setting cement.

Selenitic Cement or Selenitic Lime.—Is made by adding to the limes of the lias formation, which are the best for the purpose, or to the magnesian limestones or any lime possessing hydraulic properties, a small proportion of calcium sulphate in the form of plaster of Paris, mechanically mixed and ground with lime.

Synthesis.—The proportion of sulphate the cement contains is from 4 to 7 per cent., the usual proportion being 5. This method is most advantageously applied to the feebly hydraulic limes. When more than $7\frac{1}{2}$ per cent. of sulphate is required to suppress the slaking action, the lime is not suitable for selenitie cement, but may be rendered so by adding a lime that contains more clay.

Characteristics.—The sulphate suppresses the slaking action of lime, causing it to set more quickly, and enabling it to be used with a much larger proportion of sand than ordinary lime without loss of strength.

Uses.—For building mortar and plasterers' work.

Natural Cements.—Before Portland cement was discovered or brought to its present-day state of high efficiency, there were many natural cements, among which the most noted were Puzzolana, Roman, Medina, Atkinson's, etc. There is a great resemblance in composition and in characteristics among all these types. The following is a short description of the first two, which with slight variations, would stand for the others.

Puzzolana.—This is a volcanic substance found at Puzzola, near Naples, and in other parts of Italy, and consists of a compound of alumina and silica and traces of some of the metallic oxides, lime, potash and magnesia.

It is found, if mixed with preferably lias limes, to produce a hydraulic cement with a considerable compressional and adhesive value. This was used by Smeaton in the construction of the lighthouse built by him on the Eddystone rock. Puzzolana mortar is inferior to that made with manufactured Portland cement, and the expense of importation prohibits the use of puzzolana. An artificial puzzolana is made and largely used by grinding old well-burnt bricks and tiles, and adding to lime in lieu of sand to make mortar.

Trass is a similar material to puzzolana ; it is obtained from Andernach in Germany.

Roman cement is a natural cement prepared by burning at a low temperature nodules found in the London clay, and in the shale beds of the lias formation.

It contains about 40 per cent. of clay, is of a rich brown colour, and weighs, when ground, about 75 lbs. per bushel. It is kept in barrels, as on exposure to the atmosphere it absorbs CO_2 and moisture, and becomes inert. It should, therefore, be used fresh.

It is much weakened by the addition of sand, which should never be used in a greater ratio than 1 to 1.

It sets very rapidly, usually in about 15 minutes after mixing, and for this reason should only be mixed in small quantities as required. It was chiefly used for tidal and constructional work and where rapidity of setting was a necessity.

Portland cement has supplanted the natural varieties. Research, and the application of more scientific methods in the manufacture, both physical and chemical, have produced a cement, more consistent in its composition, more reliable in its properties, and of much greater strength than the natural cements. Increased knowledge of the treatment of Portland cement, of its various uses, and its adaptability for all purposes where the natural products would be employed, has rendered the latter obsolete in modern practice.

There are three kinds of cement of the Portland type on the market. Portland, Portland blast furnace and high alumina cement.

Portland Cement.—There are two grades of Portland; ordinary and rapid hardening. This cement is a compound of lime, silica and alumina, with a small percentage of other elements. The analysis and tests are given in the Standard Specification (see p. 20). The principal seat of the manufacture in Britain is in the neighbourhood of the mouth of the Medway, but there are in many parts of the country clay and limestones which, combined in the known proportions, produce cement to comply with the B.S.S.

The general process of manufacture is as follows. The chalk and clay in definite proportions with water, is thoroughly amalgamated in a set of washmills, and is finally conveyed to the storage tanks, where the slurry is kept in motion with mechanical stirrers. The chemical composition is tested here, and corrections made if required. The slurry is then pumped to the rotary kilns, into which it is admitted in a regulated stream.

Fig. 1 shows a section of a rotary kiln plant.

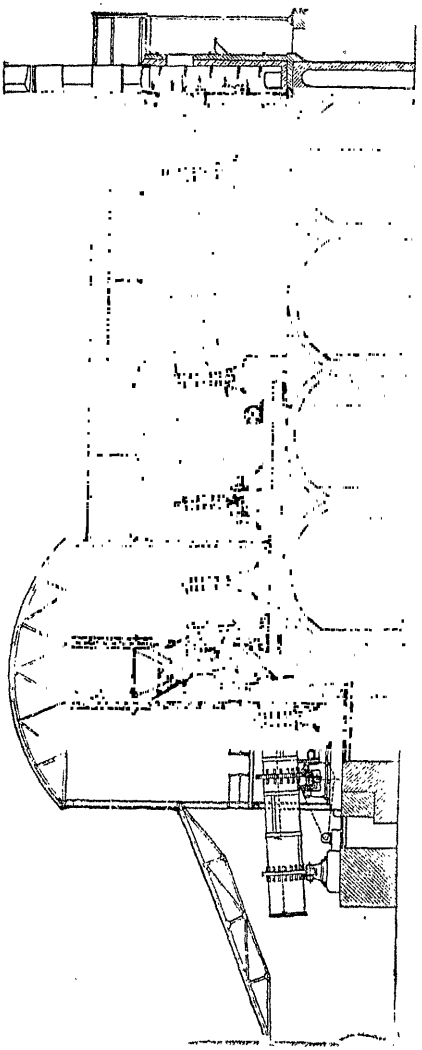


Fig. 1.—Section of Rotary Kiln Plant.

The application of the inclined rotary kiln for the purpose of drying and then calcining the dried slurry, marks undoubtedly the greatest advance in the manufacture of Portland cement. The rotary kilns are about 9 feet in diameter and 200 feet and upwards in length.

The great length of the kiln is a necessity for efficiency.

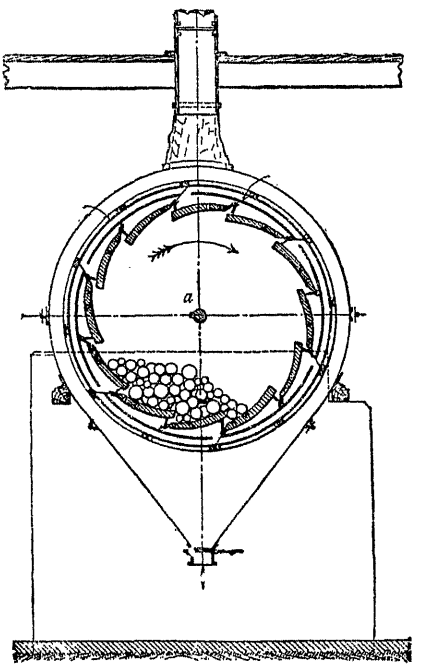


Fig. 2.—Section of Ball Mill.

The slurry enters at the upper end of the kiln and during its passage is met by the hot gases, and probably in the first 20 feet of its passage is dried, and finally leaving the kiln at its lower end, in a continuous stream of small clinker uniformly burnt, falls into the rotary coolers, which are at a lower level.

The flames at the lower end of the kiln are fed by

pulverized coal injected by means of a blast of air or steam, the zone of heat at the firing end being about 2,800° Fahr.

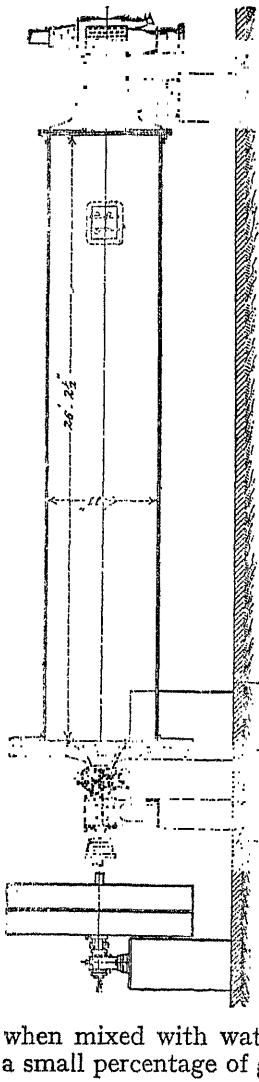


Fig. 3.—Longitudinal Section of Tube Mill.

From the rotary coolers the cement nodules are conveyed for grinding to the ball mill; a section of the latter is shown in Fig. 2, which is a specially constructed cylinder having perforated lapping plates attached and bedded to the interior surface, somewhat after the manner of roofing tiles. Inside the cylinder are a number of hardened steel balls which by the rotation of the cylinder strike the cement nodules against the steel plates and reduce the nodules to powder; as the latter is ground sufficiently fine it passes through the perforations.

The next process consists in further grinding in the tube mill, which is a cylinder charged with specially hard pebble stones from Dieppe, which grinds the cement to such fineness that it will pass through a sieve having 40,000 holes per square inch. Fig. 3 gives a longitudinal section of a tube mill, and Fig. 4 a cross-section. The more thorough the amalgamation of the raw materials the more perfect the burning, and the finer the grinding the greater will be the chemical activity of the cement when mixed with water. To obtain slow setting cements a small percentage of gypsum is sometimes added.

The setting of Portland cement is brought about by the hydration and hardening of the calcium aluminates and

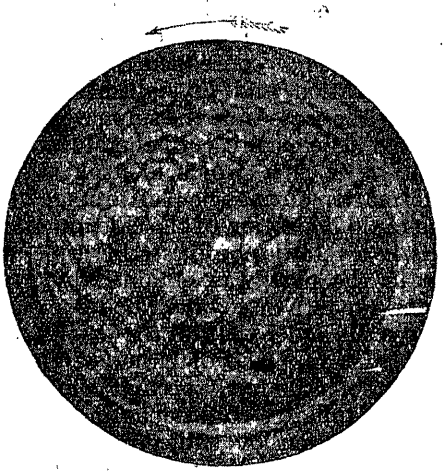


Fig. 4.—Cross-section of Revolving Tube Mill.

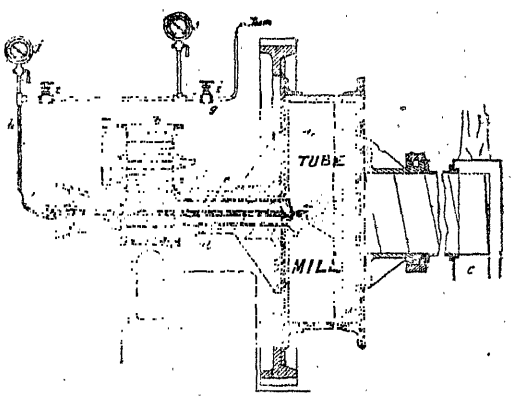


Fig. 5.—Bamber's Patent Hydration Apparatus applied to Tube Mill.

calcium silicates which constitute the bulk of Portland cement clinker.

The "initial" set of Portland cement is due to the

relatively rapid hydration and hardening of the aluminates; whilst the "final" set or hardening is primarily due to the slower chemical activity of the silicates.

In presence of gypsum the aluminates are less easily soluble, or hydrated; consequently the period within which supersaturation and crystallization take place is lengthened. At the same time the sulphate of lime (gypsum or plaster) reacts chemically with the aluminates, forming sulpho-aluminate of lime, which in itself is quick-setting, but, by its formation during the plastic stage of the mortar, prevents the rapid hydration of the calcium aluminate, and hence results in a retardation of setting time.

Aeration in the past was obtained by spreading the cement out for a month or so on a wood floor and occasionally turning over in order that all the particles may be air-slaked or cooled. In place of either of these not quite satisfactory methods the finely ground cement is subjected whilst in the tube mill to a continuous and repeated process of superficial hydration by subjecting it to a charge of superheated steam under great pressure; by this hydrating process the whole of the cement is ready for immediate use. *Fig.* 5 is a section showing the patent hydration apparatus.

The cement is finally conveyed to the machine for weighing and sacking, where it is filled into paper bags containing 112 lbs; these bags are discarded after use. For export it is stored in steel drums, which are hermetically sealed after filling.

There are six tests in the Standard Specification: (1) fineness; (2) chemical composition; (3) tensile strength when mixed with sand; (4) setting time; (5) soundness; (6) compression test on cement mortar cubes. To these rapidity in hardening should be noted, as it has an important bearing from the economic standpoint in construction.

1. *Fineness.*—It is to the extreme fineness of grinding, that the rapidity of hardening and great strength of the cement is due. The B.S.S. gives the figures for the degree of fineness.

2. *Chemical Composition.*—The S.S. gives the elements and their proportions for a good cement. The principal

constituents are lime, silica and alumina with a small percentage of sulphuric anhydride and other compounds. What actually takes place during the setting of cement is not known at present, but what probably takes place on the addition of water is that hydrates of the principal constituents are formed and harden out as calcium silicates and aluminates and the cement sets and hardens. Very divergent views are held as to the nature of the chemical reactions that take place in the setting of cement, but it is generally agreed that after the setting action has taken place there is a residue of free lime. This free lime, which readily enters into combination with many other compounds, is the probable cause of the disintegration of cement concrete in many positions.

3. *Strength Tests.*—These tests are fully explained in the Standard Specification and consist of tensile and compression tests on sand and cement briquettes, or cubes for compression; the mortar being in the proportion of 3 of sand to 1 of cement by weight.

4. *Setting Time.*—It is necessary that the setting time should be sufficient to allow the mass after mixing to be deposited and worked into position. Any disturbance after the initial set has commenced is fatal to the strength of the resulting mass. The B.S.S. gives 30 minutes as a minimum for the initial and 10 hours as a maximum for the final set. The setting action of the cement is extremely rapid, and it is usual to add in the manufacture a small percentage of gypsum to act as a retarder.

Hardness.—Apparently the final hardness and strength of the cement depends upon the extent of the perfect hydration of the finely ground particles of the clinker, hence the finer the grinding the more rapid will be the hardening. It will be seen that the setting and hardening depend upon the presence of water, therefore it is of the utmost importance that the water of hydration should not be allowed to dry out till the setting action is completed.

5. *Soundness.*—If lime is present in excess, it is liable to become overburnt in the kiln; the overburnt particles

slake very slowly and expand. This expansion may take place after the setting action is completed, causing disruption of the mass. The S.S. test is designed to accelerate the slaking of such particles, if they exist, by the application of heat, thereby discovering in a very short time a defect which would otherwise need a long period to elapse before manifesting itself by defects arising in the completed work.

Portland Blast Furnace Cement.—Iron ore contains, in addition to iron oxide, other elements and compounds such as manganese, sulphur, lime, carbon, silica and alumina. The latter two are the important compounds for cement production. In smelting the ore, coke and limestone are added, the latter as a flux. At about 1,500° Fahr. the lime enters into combination with the silica and alumina and the other impurities. It is then run off as a slag from the furnaces. On cooling it is ground similarly to Portland cement. These cements contain of the principal compounds, about 24 per cent. silica, 11 per cent. alumina and 60 per cent. lime.

There is a British Standard Specification for Portland blast furnace cement, the provisions of which are similar to those for ordinary cement.

Rapid Hardening Cement.—A serious handicap to the employment of ferroconcrete in the superstructure of a building was the length of time that was required for the concrete to harden sufficiently to enable the shuttering to be safely removed. This also led to the duplication of much of the formwork. Research work on the causes of hardening led to the evolution of rapid hardening. In this cement the strength attained in 4 days equalled that acquired by ordinary Portland in 28 days, with an ultimate strength of about one and a half that of the ordinary cement; thus allowing increased stresses. This increased resistance and rapidity of hardening is due largely to the extreme fineness of grinding. The setting time is not appreciably accelerated. The great advantage gained by the use of this cement is the considerable reduction in the time required to carry out the work and in the reduction in the cost of the shuttering. The rapidity of the hardening

enables the same formwork to be re-used several times instead of duplicating it. This effects a considerable saving in the costs.

The time that must elapse before the removal of the forms depends upon circumstances, but the following comparison indicates the advantage in this respect of the use of rapid hardening over ordinary cement.

	Ordinary Cement.	Rapid Hardening Ferrocrete.
Column, beam or girder sides ...	8 days.	1 day.
Underside of floor slabs	14 days.	2 days.
The underside of beams and girders	14 days.	3 to 6 days.

High Alumina Cement.—This is a quick setting and rapid hardening cement. Its principal components are chalk and bauxite (the ore of aluminium). The analysis of Ciment Fondu is given by the manufacturers as follows :—

SiO_2	5·95 ...	...	...	...	...	5·55
Al_2O_3	38·95 ...	...	...	...	...	36·67
Fe_2O_3	9·86 ...	...	...	...	...	11·60
FeO	6·25 ...	...	...	...	...	5·13
TiO	2·00 ...	...	...	...	...	2·08
CaO	36·25 ...	...	...	...	...	38·75

The Lafarge Aluminous Cement Co. note that considerable variations in the above percentages are possible without changing the characteristics, but that it is important that the $\frac{\text{percentage of } Al_2O_3}{\text{percentage of CaO}}$ by weight shall not be less than 0·85 or greater than 1·3. The chemical difference between Portland aluminous cement is that the principal compound of the former is a calcium silicate and of the latter a calcium aluminate.

The special properties and advantages claimed for aluminous cement are as follows: (*a*) While the setting and rapid-hardening processes of Portland depend largely upon the fineness of grinding, the same properties in aluminous cement depend mainly on its chemical composition, and that the same extreme fineness of grinding is

unnecessary. (*b*) That it does not expand on setting. (*c*) That the contraction and expansion due to temperature is the same as Portland, and that both are practically the same as steel. (*d*) The time taken for the initial set is greater than for Portland, being about 3½ hours and for the final set about 4 to 5 hours. The longer time taken for the initial set, gives more time for the manipulating and for properly placing the material. It is important that the deposited concrete be kept constantly wet for about 24 hours in order to prevent the original water of mixing from evaporating, and to take full advantage of the use of this cement. (*e*) During the period of setting and the initial stages of hardening, a great amount of heat is evolved. This renders it possible to carry on with work during frosty weather, and in temperatures that would be fatal with Portland mixtures. It is claimed that with Ciment Fondu and the ordinary concrete mix of 4.2.1 the setting and hardening action would suffer no deterioration with a temperature of 20° Fahr. below freezing, and that with richer mixes it would be possible to carry on with the temperature down to zero Fahr. During periods of high temperatures it is imperative that the concrete is maintained in a wet condition.

That aluminous cement is immune from the disintegrating or decomposing action of many substances that corrode ordinary Portland. For the standard tests, etc., on High Alumina cement, refer to B.S.S. 915.

BRITISH STANDARD SPECIFICATION FOR ORDINARY PORTLAND AND RAPID-HARDENING PORTLAND CEMENTS

(REVISED JULY, 1940) No. 12—1940

NOTE.—*The Institution desires to call attention to the fact that this Specification is intended to include the technical provisions necessary for the supply of the material herein referred to, but does not purport to include all the necessary provisions of a contract.*—

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advisable to obtain the latest official copy from the British Standards Institution, 28, Victoria Street, London, S.W.1.

NOTE.—The term "Rapid-hardening" has been adopted in this Specification because its use has become accepted in this country. It is synonymous with the designation "High Early Strength" which is used in other countries, and should not be confused with "Quick-setting." It will be seen from Clause 12 that rapid-hardening cement (unless particularly specified or required) is not quick-setting.

1. *Composition and Manufacture of Cement.*—The cement, whether ordinary or rapid-hardening, shall be manufactured by intimately mixing together calcareous and argillaceous and/or other silica, alumina or iron oxide bearing materials, burning them at a clinkering temperature and grinding the resulting clinker so as to produce a cement capable of complying with this Specification.

No addition of any material shall be made after burning other than calcium sulphate, or water, or both.*

2. *Samples for Testing and by whom to be taken.*—A sample or samples for testing may be taken by the Purchaser or his representative, or by any person appointed to superintend the works for the purpose of which the cement is required, or by his representative, or by any expert analyst employed or instructed by such Purchaser or person, or by the representative of such Purchaser or person.

3. *Samples for Testing and how to be taken.*—Each sample for testing shall consist of a mixture of approximately equal portions selected from at least twelve different positions in the heap or heaps when the cement is loose, or from not less than twelve different bags, barrels or other packages, when the cement is not loose, or where there is a less number than twelve different bags, barrels or other packages, then from each bag, barrel or other package. Every care shall be taken in the selection, so that a fair average sample shall be taken. Such final sample shall weigh at least 10 lbs. (4.54 kg.).

* No cement to which slag has been added or which is a mixture of Portland Cement and slag shall be deemed to comply with this Specification. For a Specification which admits of added slag *see* B.S. 146. Portland-Blast-furnace Cement.

4. *Sampling Large Quantities.*—When more than 250 tons (560,000 lbs. = 254,012 kg.) of cement is to be sampled at one time, separate samples shall be taken, as provided in Clause 3, from each 250 tons or part thereof.

Not more than 250 tons shall be stored in such a manner that it cannot be separately identified and sampled in accordance with the provisions of this clause and of Clause 3, and separated in bulk from the remainder. If more than 250 tons of cement is stored in a silo, provision shall be made by which each 250 tons, or any part of 250 tons in excess thereof, shall be isolated from the remainder and sampled at different points.*

5. *Facilities for Sampling and Identifying.*—The Vendor† shall afford every facility, and provide all labour and materials, for taking and packing the samples for testing the cement and for subsequently identifying the cement sampled.

6. *Cost of Tests, Analyses and Samples.*—The tests and chemical analyses hereinafter mentioned below, other than those referred to in Clause 15, shall (unless otherwise provided in the contract between the Vendor and the Purchaser) be made at the expense of the Purchaser, but no charge shall be made by the Vendor for the cement used for samples or for carriage thereon.

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* As there are silos in existence with capacities greater than 250 tons and which cannot be sub-divided without danger to the structure, sampling from such silos shall be permitted provided the Purchaser or his representative agrees and is satisfied that a proper and representative sample can be obtained of each 250 tons discharged into the silo. Such samples can be obtained either from suitable sampling holes in the walls of the silo or by automatic means at the point of discharge into the silo.

In the event of any such sample representing a 250-ton portion not complying with the requirements of this specification, the Purchaser or his representative may refuse to accept any cement from the particular silo from which the sample was drawn.

It is, however, the intention of this specification that wherever possible each 250 tons shall be isolated.

† The term "Vendor" throughout this specification shall mean the seller of the cement, whether he be the manufacturer of the cement or not.

7. *Tests.*—The sample or samples shall be tested in the manner mentioned below for :—

(*a*) Fineness.
(*b*) Chemical composition.
(*c*) Strength.
(*d*) Setting time.
(*e*) Soundness.

8. *Test for Fineness.*—The cement shall comply with the following conditions of fineness :—100 g. (or say 4 ozs.) of cement shall be continuously sifted for a period of 15 minutes on a B.S. Test Sieve No. 170 with the following results :—

Ordinary Portland Cement.
The residue, by weight, shall not exceed 10 per cent.

Rapid-hardening Portland Cement.
The residue, by weight, shall not exceed 5 per cent.

Air-set lumps in the samples may be broken down with the fingers, but nothing shall be rubbed on the sieve.

The sieves shall be prepared from wire cloth complying with the requirements of Table 1 of B.S. 410—1931, Test Sieves. The wire cloth shall be woven (not twilled) and carefully mounted on the frames without distortion. The sieving surface shall be not less than 50 sq. in. (322·58 sq. cm.) and the depth of the sieves shall be not less than $2\frac{3}{4}$ ins. (69·85 mm.) measured from the surface of the wire cloth.

The nominal dimensions and tolerances for wire cloth for sieves for testing cements are given in the table below :—

TABLE 1.—DIMENSIONS OF STANDARD WIRE CLOTH FOR SIEVES FOR TESTING CEMENT.

B.S. Mesh No. (Nominal meshes per linear inch).	Nominal size of aperture (side of square).		Nominal diameter of wire.			Approx. screening area.	Tolerance on average aperture plus or minus.
	in.	mm.	in.	mm.	Standard Wire Gauge.	per cent.	per cent.
170	0·0035	0·089	0·0024	0·061	46	35	8

The maximum tolerances for occasional large apertures, if present, expressed as percentages of the nominal dimensions for side of aperture in either direction shall not exceed 50 per cent.

No sieve shall be regarded as Standard which does not conform to these requirements.

9. *Test for Chemical Composition.*—The cement shall comply with the following conditions as to its chemical composition. The percentage of lime, after deduction of that necessary to combine with the sulphuric anhydride present shall be not more than 2·8 times the percentage of silica plus 1·2 times the percentage of alumina, plus 0·65 times the percentage of iron oxide, nor be less than two-thirds of that amount. The ratio of the percentage of iron oxide to that of alumina shall not exceed 1·5. The weight of insoluble residue shall not exceed 1·0 per cent., that of magnesia shall not exceed 4 per cent., and the total sulphur content calculated as sulphuric anhydride (SO_3) shall not exceed 2·75 per cent. The total loss on ignition shall not exceed 3 per cent. for cement manufactured or sampled or tested in temperate climates and 4 per cent. for cement manufactured or sampled or tested in hot climates.

10. *Determination of Normal Consistency of Cement Paste.*—Cement paste of normal consistency shall be used in the tests for soundness and setting time, and also as a guide to the amount of water required for the gauging of cement and sand briquettes for the test for ultimate tensile stress as described in Clause 11.

For the purpose of arriving at the normal consistency of cement paste the Vicat apparatus, shown on page 40, shall be used, the plunger (**G**), 1 cm. in diameter, being substituted for the needle there shown in position.

The quantity of water required to produce a paste of normal consistency shall be 0·78 of that required to give a paste which will permit of the settlement of the Vicat plunger to a point 5 to 7 mm. from the bottom of the Vicat mould when the cement paste is tested as described below.*

* To facilitate this computation a table Table 3) is given on page 35.

The time of gauging, that is, the time elapsing from the moment of adding the water to the dry cement until commencing to fill the mould, shall be not less than 3 minutes nor more than 5 minutes. Where a quick-setting cement has been specially specified or required, the time of gauging shall be not less than 2 minutes nor more than 3 minutes and the filling of the mould shall be completed within 5 minutes. In either case the gauging shall be completed before signs of setting occur.

The cement paste is filled into the Vicat mould (E), page 40, the mould resting upon a non-porous plate. The mould shall be completely filled and the surface of the paste shall then be smoothed off level with the top of the mould.

In filling the mould the operator's hands and the blade of the ordinary gauging trowel shall alone be used. The trowel shall weigh about 7½ ozs. (213 g. approx.). The mould after being filled may be lightly shaken to the extent necessary for expelling the air.

Clean appliances shall be used for gauging and the temperature of the cement and water and that of the test room at the time when the above operations are being performed shall be from 58° to 64° Fahr. (14·4° to 17·8° Cent.), subject to the provisions of Clause 17.

The test block confined in the mould and resting on the plate shall be placed under the rod bearing the plunger, the latter shall then be lowered gently into contact with the surface of the test block and quickly released and allowed to sink into the same.

Trial pastes shall be made up of varying percentages of water until the amount necessary for determining the normal consistency as defined above is found. This amount of water used shall be recorded and expressed as a percentage by weight of the dry cement.

11. *Tests for Strength* (*Cement and Sand*). (*a*) *Ultimate Tensile Stress.*—The ultimate tensile stress of cement and sand shall be ascertained from briquettes of the shape shown on page 38. The briquettes shall be prepared in the following manner :—

Preparation of Briquettes.—A mixture of cement and sand in the proportion of 1 part by weight of cement to 3 parts by weight of the standard sand specified on

page 27 shall be gauged with water, the percentage of water to be used being determined by the following formula :—

$$\tfrac{1}{4} P + 2{\cdot}50$$

where P is the percentage of water required to produce a paste of normal consistency determined as described in Clause 10.*

The mixture gauged as above, shall be evenly distributed in moulds of the form required to produce briquettes of the shape shown on page 38, each mould resting upon a non-porous plate. After filling a mould, a small heap of the mixture shall be placed upon that in the mould and beaten down with the standard spatula shown on page 39, until the mixture is level with the top of the mould. This last operation shall be repeated on the other side and the mixture beaten down until water appears on the surface ; the flat only of the standard spatula is to be used, and no other instrument or apparatus is to be employed for this operation. The briquettes shall be finished off in the moulds by smoothing the surface with the blade of a trowel.

Clean appliances shall be used for gauging, and the temperature of the water and that of the test room at the time when the above operations are being performed shall be from 58° to 64° Fahr. (14·4° to 17·8° Cent.), subject to the provisions of Clause 17.

The briquettes shall be kept at a temperature of 58° to 64° Fahr. (14·4° to 17·8° Cent.) in an atmosphere of at least 90 per cent. relative humidity for 24 hours after gauging, when they shall be removed from the moulds and immediately submerged in clean fresh water, and left there until taken out just prior to breaking. The water in which they are submerged shall be renewed every seven days and maintained at a temperature of between 58° and 64° Fahr. (14·4° to 17·8° Cent.), subject to the provisions of Clause 17. After they have been so taken out and until they are broken the briquettes shall not be allowed to become dry.

Breaking.—The briquettes shall be tested for ultimate tensile stress at the periods after gauging mentioned

* To facilitate this computation a table (Table 3) is given on page 35.

below. Six briquettes shall be tested at each period and the ultimate tensile stress shall be the average ultimate tensile stress of the six briquettes for such period. The briquettes to be tested shall be held in strong metal jaws of the shape shown on page 38,* and the load steadily and uniformly applied, starting from zero, and increased at the rate of 100 lbs. per square inch of section (7.03 kg. per sq. cm.) in 12 seconds.

The ultimate tensile stress of the briquettes shall be as follows :—

Ordinary Portland Cement.

3 days (72 hours)	Not less than 300 lbs. per square inch (21·09 kg. per sq. cm.).
7 days	Shall show an increase on the ultimate tensile stress at 3 days and be not less than 375 lbs. per square inch (26·37 kg. per sq. cm.).

Rapid-hardening Portland Cement.

1 day (24 hours)	Not less than 300 lbs. per square inch (21·09 kg. per sq. cm.).
3 days (72 hours)	Shall show an increase on the ultimate tensile stress at 1 day and be not less than 450 lbs. per square inch (31·63 kg. per sq. cm.).

Standard Sand.—The standard sand shall be obtained from Leighton Buzzard, shall be of the white variety and shall be thoroughly washed and dried. Its loss of weight on extraction with hot hydrochloric acid shall be not more than 0·25 per cent.†

* In order to distribute the stress set up by the pressure of the jaws over as large a surface of the briquette as possible, it is recommended that rubber or greased paper be inserted between the sides of the briquette and the jaws of the machine.

† To carry out the test, dry the sand at 100° Cent. for 1 hour, weigh out 2 g. into a porcelain dish, add 20 ml. of hydrochloric acid of specific gravity 1·16 and 20 ml. of distilled water. Heat on a water bath for 1 hour, filter, wash well with hot water, dry and ignite in a covered crucible.

The sand shall pass through a B.S. Test Sieve No. 18, and be retained on a B.S. Test Sieve No. 25. The sieves shall be prepared from wire cloth complying with the requirements of Table I of B.S. 410—1931, Test Sieves. The wire cloth shall be woven (not twilled) and carefully mounted on the frames without distortion. The sieving surface shall be not less than 50 square inches (322·58 sq. cm.) and the depth of the sieves shall be not less than $2\frac{3}{4}$ inches (69·85 mm.) measured from the surface of the wire cloth.

The nominal dimensions and tolerances for wire cloths for sieves for preparing standard sand are given in the table below :—

TABLE 2.—DIMENSIONS OF STANDARD WIRE CLOTHS FOR SIEVES FOR PREPARING STANDARD SAND.

B.S. Mesh No. (Nominal meshes per linear inch).	Nominal size of Aperture (side of square).		Nominal diameter of Wire.			Approx. Screening Area.	Tolerance on Average Aperture plus or minus.
	in.	mm.	in.	mm.	Standard Wire Gauge.	per cent.	per cent.
25	0·0236	0·599	0·0164	0·417	27	35	5
18	0·0336	0·853	0·022	0·559	24	36	5

The maximum tolerances for occasional large apertures, if present, expressed as percentages of the nominal dimensions for side of aperture in either direction shall not exceed 20 per cent.

No sieve shall be regarded as Standard which does not conform to these requirements.

(*b*) *Compressive Strength.*—When requested by the Purchaser the following test for compressive strength (cement and sand) shall be substituted for the test for ultimate tensile stress (cement and sand) described above.

The compressive strength of cement and sand shall be ascertained from cubes having a length of side of 2·78

inches * (the area of face equals 50 sq. cm.). The cubes shall be prepared in the following manner :—

Preparation of Cubes.—A mixtute of cement and sand in the proportion of 1 part by weight of cement to 3 parts by weight of the Standard Sand specified on page 27 shall be mixed dry with a trowel on a non-porous plate for 1 minute and then with water until the mixture is of a uniform colour, the percentage of water being 10 per cent. by weight of the dry materials. Should the time taken to obtain a uniform colour exceed 4 minutes the mixture shall be rejected and the operation repeated with a fresh quantity of cement, sand and water.

The material for each cube shall be mixed separately and the quantities of cement, sand and water, for both Ordinary Portland Cement and Rapid-hardening Portland Cement shall be as follows :—

Cement	185 g.
Sand	555 g.
Water	74 g.

Immediately after the mixing, the mortar shall be placed in the hopper of the cube mould in one portion and shall be compacted by vibration.

The vibration machine shall be constructed as described in Appendix A and shown facing page 42, and shall have a frequency of 12,000 vibrations per minute (± 400 v.p.m.) with an amplitude of 0·0022 inch. The mould shall be of metal accurately machined and with parallel faces and together with its base plate shall be firmly held on the table of the vibration machine. A hopper of suitable size and shape shall be securely attached to the mould to facilitate filling and this hopper shall not be removed until completion of the vibrating period. A suitable mould and hopper are shown on pages 42 and 43 .

The mould filled with mortar shall be vibrated for 2 minutes at full speed and shall not otherwise be finished

* When so desired, 3 inch cubes may be used for the test, in which case the quantities shall be as follows :—

Cement	235 g.
Sand	705 g.
Water	94 g.

by trowel or by hand. The faces and joints of the mould shall be covered with a film of petroleum jelly and the joints shall be tightly made so as to ensure that no water escapes during the vibration.

Clean appliances shall be used for mixing and the temperature of the water and that of the test room at the time when the above operations are being performed shall be from 58° to 64° Fahr. (14·4° to 17·8° Cent.).

The cubes shall be kept at a temperature of 58° to 64° Fahr. (14·4° to 17·8° Cent.) in an atmosphere of at least 90 per cent. relative humidity for 24 hours after completion of vibration, when they shall be removed from the moulds and immediately submerged in clean, fresh water, and left there until taken out just prior to breaking. The water in which they are submerged shall be renewed every 7 days and maintained at a temperature of between 58° and 64° Fahr. (14·4° to 17·7° Cent.). After they have been so taken out and until they are broken, the cubes shall not be allowed to become dry.

Crushing.—The cubes shall be tested for compressive strength at the periods mentioned below, which periods shall be reckoned from the completion of vibration. Three cubes shall be tested at each period and the compressive strength shall be the average compressive strength of the three cubes for such period.

Ordinary Portland Cement.

3 days (72 hours) and 7 days respectively.

Rapid-hardening Portland Cement.

1 day (24 hours) and 3 days (72 hours) respectively.

The cubes shall be tested on their sides without any packing between the cube and the steel platens of the testing machine. One of the platens shall be carried on a ball and shall be self-adjusting and the load shall be steadily and uniformly applied starting from zero at a rate of 5,000 lbs. per square inch per minute. The compressive strength shall be calculated from the crushing load and the average area over which the load is applied.

The compressive strength of the cubes shall be as follows :—

Ordinary Portland Cement.

3 days (72 hours)	Not less than 1,600 lbs. per square inch (112·5 kg. per sq. cm.).
7 days	Shall show an increase on the compressive strength at 3 days and be not less than 2,500 lbs. per square inch (175·8 kg. per sq. cm.).

Rapid-hardening Portland Cement.

1 day (24 hours)	Not less than 1,600 lbs. per square inch (112·5 kg. per sq. cm.).
3 days (72 hours)	Shall show an increase on the compressive strength at 1 day and be not less than 3,500 lbs. per square inch (246·1 kg. per sq. cm.).

Standard Sand.—The standard sand shall comply with the requirements specified in Clause 11 (*a*), Ultimate Tensile Stress.

12. *Tests for Setting Time. Vicat Apparatus.* The initial and final setting times of the cement shall be determined by means of the Vicat apparatus shown on page 40.

For the purpose of carrying out the tests, a test block shall be made as follows :—

Neat cement shall be gauged in the manner and under the conditions prescribed in Clause 10. The test block shall be made by filling the cement, gauged as above, into the Vicat mould (E), on page 40, the mould resting upon a non-porous plate. The mould shall be completely filled, and the surface of the paste shall then be smoothed off level with the top of the mould.

Clean appliances shall be used for gauging, and the temperature of the water and that of the test room at the time when the above operations are being performed shall be from 58° to 64° Fahr. (14·4° to 17·8 Cent.), subject to the provisions of Clause 17.

The test block shall be kept during the whole time of the test at a temperature of 58° to 64° Fahr. (14·4° to 17·8° C.) in an atmosphere of at least 90 per cent. relative humidity and away from draughts.

Determination of Initial Setting Time.—For the determination of the initial setting time the test block confined in the mould and resting on the plate shall be placed under the rod bearing the needle (C); the latter shall then be lowered gently into contact with the surface of the test block and quickly released, and allowed to sink into the same. This process shall be repeated until the needle, when brought into contact with the test block and released as above described, does not pierce it completely. The period elapsing between the time when the water is added to the cement and the time at which the needle ceases to pierce the test block completely shall be the initial setting time above referred to.

Determination of Final Setting Time.—For the determination of the Final Setting time the needle (C) of the Vicat apparatus shall be replaced by the needle with an annular attachment (F), shown separately on page 40. The cement shall be considered as finally set when, upon applying the needle gently to the surface of the test block, the needle makes an impression thereon, while the attachment fails to do so. In the event of a scum forming on the surface of the test block, the underside of the test block may be used for determining the final set.

Normal-setting Cement.—Unless a quick-setting cement is particularly specified or required, the initial setting time of the cement shall be not less than 30 minutes and the final setting time not more than 10 hours.

Quick-setting Cement.—If a quick-setting cement is particularly specified or required, the initial setting time of the cement shall be not less than 5 minutes, and the final setting time not more than 30 minutes.

13. *Test for Soundness.*—The cement shall be tested for soundness by the "Le Chatelier" method. The apparatus for conducting the "Le Chatelier" test is shown on page 41.

The moulds shall be kept in good condition, having the jaws not more than 0·5 mm. (0·02 inch) apart.

In conducting the test the mould shall be placed upon a small piece of glass and filled with cement paste of normal consistency gauged in the manner and under the conditions prescribed in Clause 10, care being taken to keep the edges of the mould gently together whilst this operation is being performed. The mould shall then be covered with another glass plate, upon which a small weight shall be placed, and the whole shall then be immediately submerged in water at a temperature of 58° to 64° Fahr. (14·4° to 17·8° Cent.), subject to the provisions of Clause 17, and left there for 24 hours.

The distance separating the indicator points shall then be measured, and the mould again submerged in water at the temperature prescribed above, which shall be brought to boiling point in 25 to 30 minutes and kept boiling for 3 hours. The mould shall then be removed from the water and allowed to cool and the distance between the points again measured; the difference between the two measurements represents the expansion of the cement and shall not exceed 10 mm. (0·40 inch). In the event of the cement's failing to comply with this test, a further test shall be made from another portion of the same sample after it shall have been aerated by being spread out to a depth of 76·20 mm. (3 inches) at the temperature prescribed above for a total period of 7 days, when the expansion, determined as above, shall not exceed 5 mm. (0·20 inch).

14. *Non-compliance with Tests.*—Any cement which does not comply with the whole of the tests and analyses specified above, or which has not been stored in the manner provided in Clause 4, may be rejected as not complying with this Specification.

15. *Copies of Vendor's Tests, Analyses, etc.*—The Vendor shall, if required, furnish free of cost a copy of any document in his possession showing the result of any tests or analyses made for him, or for any other person, of any cement sold or offered for sale to the Purchaser or of the lot from which the cement so sold or offered for sale has been or is to be taken. The Vendor shall, if required at the

time of purchase, furnish, free of cost, a certificate that the cement so sold or offered for sale has been tested and analysed, and that such tests and analysis comply in all respects with this Specification. The furnishing of such copies of documents or the giving of such certificate shall not preclude the Purchaser from rejecting any cement which does not comply with this Specification.

16. *Delivery.*—Unless otherwise agreed between the Purchaser and the Vendor the cement shall be packed in bags containing 112 lbs. (50·8 kg.) net or in wooden casks or steel drums containing 375 lbs. (170·1 kg.) net bearing the Manufacturer's name or registered mark. These weights shall be legibly marked upon the packages and shall indicate the contents within reasonable limits of accuracy.

17. *Cement in Hot Climates.*—The temperatures specifically mentioned in Clauses 10, 11 (*a*) and (*b*), 12 and 13 are applicable to temperate climates. For cement intended for use in hot climates the cement may be tested at any higher temperature up to 95° Fahr. (35° Cent.) and shall comply with the requirements of this Specification.*

* When cement is tested at temperatures above 58° to 64° Fahr. (14·4° to 17·8° Cent.) the setting time and strength requirements may be altered if agreed between Purchaser and Vendor. It should be noted that an increase in the testing temperature reduces the setting time and increases both the ultimate tensile stress and the compressive strength.

TABLE 3.—PERCENTAGES OF WATER FOR GAUGING CEMENT PASTE AND 3 TO 1 STANDARD SAND AND CEMENT MORTAR.

Percentage of Water			Percentage of Water		
required to give a Standard penetration of 5 to 7 mm. from bottom of Vicat mould.	to be used for gauging Cement Paste (Normal Consistency).	to be used for gauging 3 to 1 Standard Sand and Cement Mortar.	required to give a Standard penetration of 5 to 7 mm. from bottom of Vicat mould.	to be used for gauging Cement Paste (Normal Consistency).	to be used for gauging 3 to 1 Standard Sand and Cement Mortar.
per cent.	per cent.	per cent.	per cent.	per cent.	per cent.
33·0	25·7		31·2	24·3	
32·9			31·1		8·6
32·8	25·6	8·9	31·0	24·2	
32·7	25·5		30·9	24·1	
32·6	25·4		30·8	24·0	
32·5			30·7	23·9	8·5
32·4	25·3		30·6		
32·3	25·2		30·5	23·8	
32·2	25·1	8·8	30·4	23·7	
32·1	25·0		30·3	23·6	
32·0			30·2		8·4
31·9	24·9		30·1	23·5	
31·8	24·8		30·0	23·4	
31·7	24·7	8·7	29·9	23·3	
31·6	24·6		29·8	23·2	
31·5			29·7		8·3
31·4	24·5	8·6	29·6	23·1	
31·3	24·4		29·5	23·0	

TABLE 3 (*continued from page* 35).

Percentage of Water			Percentage of Water		
required to give a Standard penetration of 5 to 7 mm. from bottom of Vicat mould.	to be used for gauging Cement Paste (Normal Consistency).	to be used for gauging 3 to 1 Standard Sand and Cement Mortar.	required to give a Standard penetration of 5 to 7 mm. from bottom of Vicat mould.	to be used for gauging Cement Paste (Normal Consistency).	to be used for gauging 3 to 1 Standard Sand and Cement Mortar.
per cent.	per cent.	per cent.	per cent.	per cent.	per cent.
29·4	22·9		27·6	21·5	
29·3			27·5		7·9
29·2	22·8	8·2	27·4	21·4	
29·1	22·7		27·3	21·3	
29·0	22·6		27·2	21·2	
28·9	22·5		27·1	21·1	7·8
28·8			27·0		
28·7	22·4	8·1	26·9	21·0	
28·6	22·3		26·8	20·9	
28·5	22·2		26·7	20·8	
28·4			26·6	20·7	7·7
28·3	22·1		26·5		
28·2	22·0		26·4	20·6	
28·1	21·9	8·0	26·3	20·5	
28·0	21·8		26·2	20·4	7·6
27·9			26·1		
27·8	21·7	7·9	26·0	20·3	
27·7	21·6				

APPENDIX A

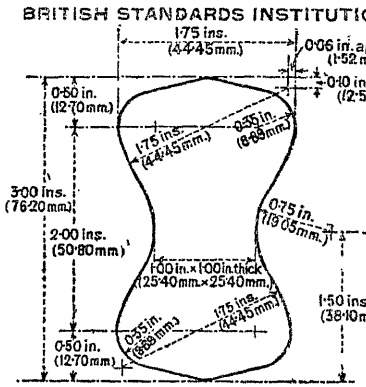
DESCRIPTION OF VIBRATION MACHINE FOR COMPACTING MORTAR CUBES FOR COMPRESSIVE STRENGTH TEST

The vibration machine consists of a frame mounted on light springs to carry the cube mould, and a revolving shaft provided with an eccentric. By means of a balance weight beneath the base plate attached rigidly to the frame, the centre of gravity of the whole machine, including the cube and mould, is brought to the centre of the eccentric shaft. In consequence of this, the revolving eccentric imparts an equal circular motion to all parts of the machine and mould, the motion being equivalent to equal vertical and horizontal simple harmonic vibrations 90° out of phase. The minimum running speed of the machine is well above its natural frequency on its supporting springs, so that the amplitude of vibration is independent of the speed.

The machine shall be constructed to comply with the following requirements :—

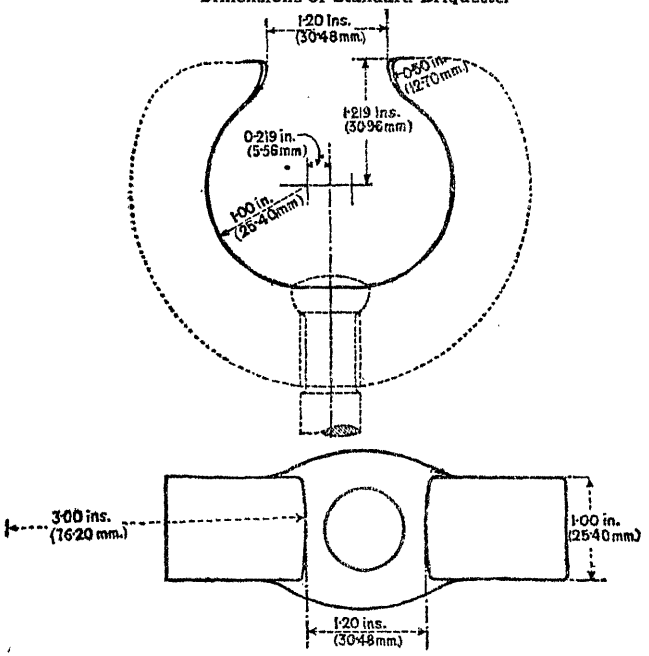
Total weight of machine on its supporting springs (excluding weight of solid eccentric but including weight of hopper, mould and cube)...	= 64 lbs. approx.
Weight of solid eccentric	= 0·7 lbs. ,,
Eccentricity of eccentric	= 0·2 in.
Normal running speed of eccentric shaft	= 12,000 (± 400) r.p.m.

A machine of the type described above is shown facing p. 42, and a mould, clamp, and hopper suitable for use with the machine are shown separately on pages 42 and 43.



Briquette to have a uniform thickness of 1 in. throughout.

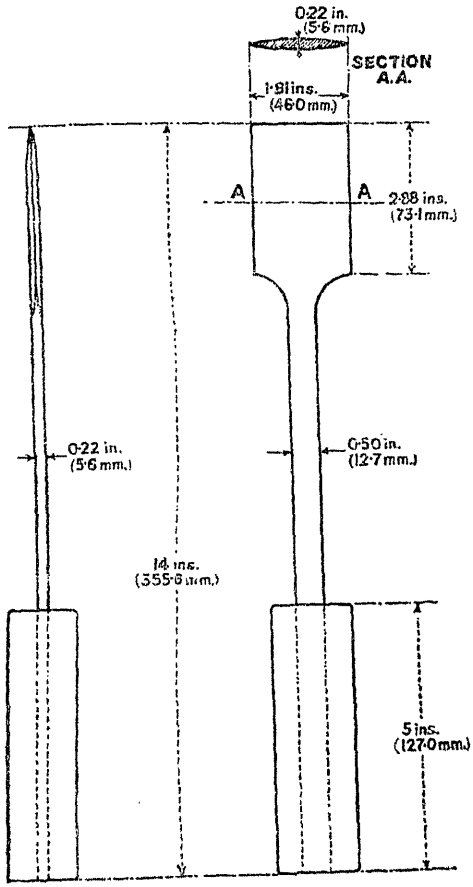
Dimensions of Standard Briquette.



Elevation and plan of jaws for holding briquette.

NOTE.—*The figures in brackets are approximate equivalents.*

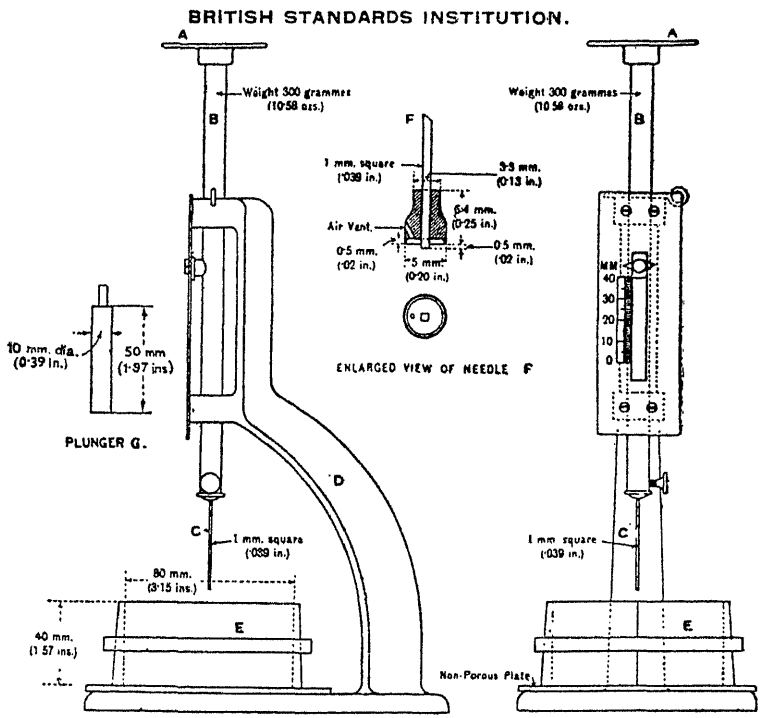
BRITISH STANDARDS INSTITUTION.



The standard spatula shown above is of steel to which a wooden handle is securely attached. The total weight shall not exceed 12 oz. (340 g.), and the centre of gravity shall fall within 0·25 in. (6·35 mm.) of the centre of the length of the spatula.

Standard Spatula.

NOTE.—*The figures in brackets are approximate equivalents.*



The Vicat Apparatus consists of a frame (**D**) bearing a movable rod (**B**) with, at one end, the cap (**A**) and at the other, one of the following which are removable:—(*a*) The needle (**C**) for determining the initial setting time, (*b*) the needle (**F**) for determining the final setting time or (*c*) the plunger (**G**) for determining the normal consistency.

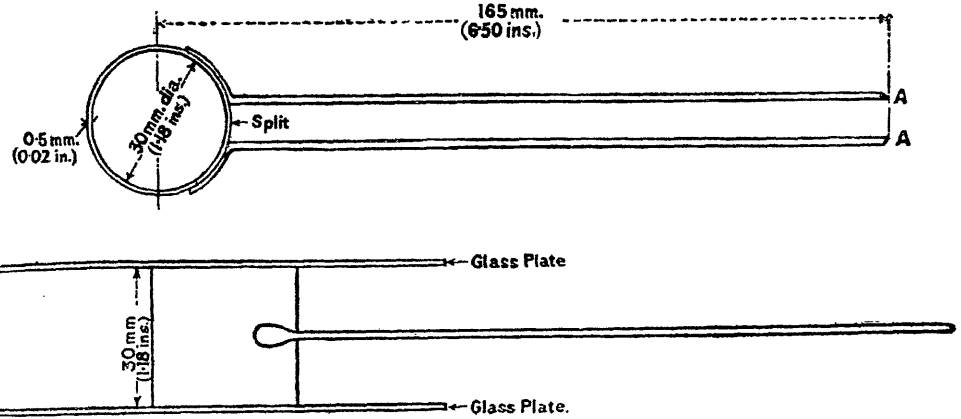
The needle (**C**) shall be 1 mm. (0·039 in.) square in section and have a flat end. The needle (**F**) shall be of the same shape and section as needle (**C**) but shall be fitted with a metal attachment hollowed out so as to leave a circular cutting edge 5 mm. (0·20 in.) in diameter, the end of the needle projecting 0·5 mm. (0·02 in.) beyond this edge. The plunger (**G**) shall be of polished brass 10 mm. (0·39 in.) in diameter, 50 mm. (1·97 in.) long with a projection at the upper end for insertion into the movable rod (**B**) and the lower edge shall be flat. The movable rod (**B**) carries an indicator which moves over a graduated scale attached to the frame (**D**).

With all attachments, the cap and rod, with needle (**C**) or needle (**F**), or plunger (**G**), shall together weigh 10·58 oz. (300 g.).

The mould for the cement consists of a split ring (**E**) 80 mm. (3·15 in.) in diameter, 40 mm. (1·75 in.) high, which rests on a non-porous plate.

Vicat Apparatus for determining the normal consistency and setting time of cement.

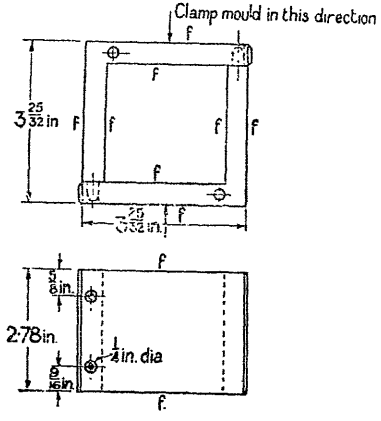
NOTE.—*The figures in brackets are approximate equivalents.*



The apparatus for conducting the " Le Chatelier " test consists of a small split cylinder of spring brass or other suitable metal of 0·5 mm. (0·02 in.) in thickness, forming a mould 30 mm. (1·18 in.) internal diameter and 30 mm. (1·18 in. high). On either side of the split are attached two indicators with pointed ends **AA**., the distance from these ends to the centre of the cylinder being 165 mm. (6·50 in.).

Apparatus for conducting the "Le Chatelier" test.

NOTE.—*The figures in brackets are approximate equivalents.*



MOULD

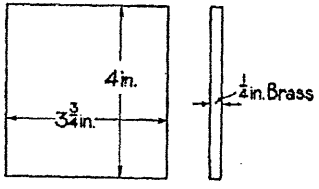
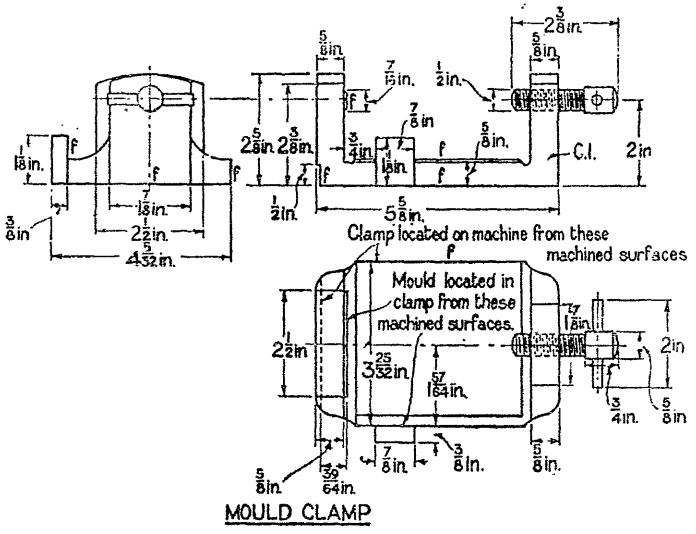
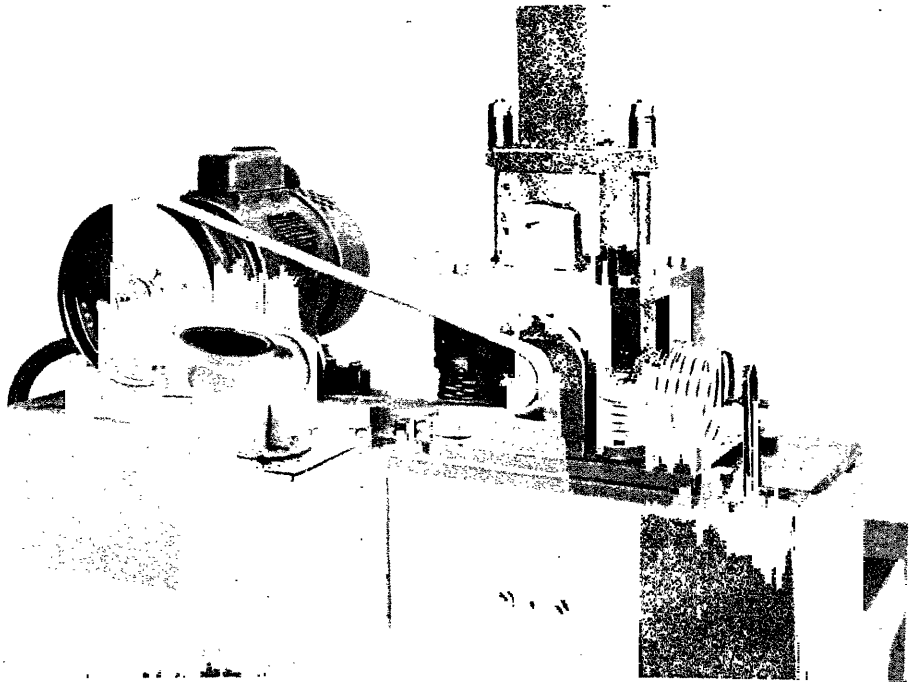


PLATE TO FORM BOTTOM OF MOULD

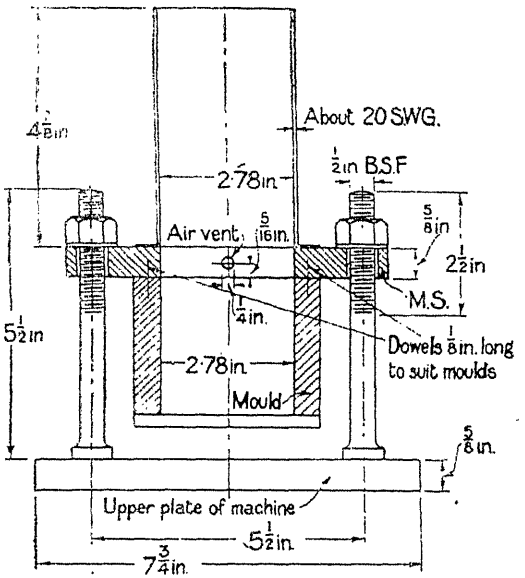


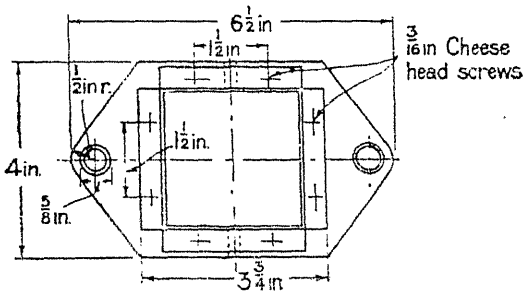
MOULD CLAMP



Vibration Machine for compacting Mortar Cubes for the compressive strength test.

[To face p. 42.


4⅝in
About 20 SWG.
½in B.S.F
2·78in
Air vent
5/16in.
⅝in
2½in
M.S.
¼in.
5½in
Dowels ⅛in. long to suit moulds
2·78in.
Mould
⅝in.
Upper plate of machine
5½in.
7¾in.


6½in
1½in
3/16in Cheese head screws
½in r.
1½in.
4in.
⅝in.
3¾in.

HOPPER

CHAPTER II

PLASTERING

Definition.—Surfaces covered by any calcareous compound are said to be plastered. External work is usually completed in two coats, whilst internal plastering is more often effected in three layers. The first coat placed against a brick or stone wall is called rendering ; when applied over wood laths or wire netting it is termed pricking up : the second or intermediate is called the floating coat, and the third or finishing, the setting coat, and generally any calcareous covering, whether put on in one, two, or three coats, is known as plastering.

Materials.—There are a number of plasters employed chiefly for internal work derived from calcium sulphate $CaSO_4$. This is found as a mineral anhydrite and hydrated as gypsum $CaSO_4 2H_2O$. These plasters are on the market in two forms. The anhydrous and the hemihydrates. The best known of the former type are the Keene's and Parian cements. The anhydrous plasters are prepared by heating gypsum at a high temperature to drive off all the water, giving $CaSO_4$. In this condition the material would be extremely slow setting. During the calcining process it is mixed with a small percentage of an accelerator. According to the original methods of preparation alum was added to the anhydrate to form Keene's and borax to produce Parian. Though these names survive the original modes of manufacture are not necessarily adhered to. When mixed with water, the initial set takes from 2 to 10 hours and the final about 16.

Both Keene's and Parian set very hard and can be worked up to a polished surface. Keene's is much used for angles and exposed portions of walls or surfaces liable

to injury. Where surfaces are to be finished in either of these cements the first coat should be in Portland cement. The second coat in a coarse variety of Keene's or Parian. These cements are commonly marketed in three grades of fineness: coarse, fine, and superfine, the latter being used for the finishing coat. The cements are eminently suitable for all kinds of decorative work and are indispensable for the manufacture of artificial marbles. For the latter purpose they should be neither acid nor alkaline, or the colouring of the veinings will be affected. A valuable property of these cements is that they can be painted within a few hours of finishing.

Hemihydrates.—There are a large number of plasters of this description on the market having gypsum for a base. The gypsum $CaSO_4 . 2H_2O$ is heated up to about 140° Cent. Some of the water is expelled. The resulting compound is $2CaSO_4 . H_2O$. This is ground into a fine white powder known commercially as plaster of Paris. When water is added to this powder it takes up the amount it lost when heated and sets rapidly into a hard mass, the time taken being about 5 minutes. This setting time is much too rapid for general work, but it is a valuable medium for casting ornament and for gauging with lime putty for running mouldings, etc.

Hardwall Plasters.—These are hemihydrates to which a retarder has been added to increase the time of setting. The retarders employed are substances of a colloidal nature; their action seems to be to delay crystallization. The addition of small percentages does not seem to affect the strength of the compound, but if used in excess, the strength is diminished. There are a large number of these hardwall plasters marketed, there are slight variations in their properties, most of them are proprietary articles, and for the best results, the makers' advice should be obtained and their instructions followed. These plasters, owing to the retarded set, can be employed for large surfaces and give hard smooth finishing coats. It is unnecessary to use Keene's or any other hard cements for salient angles, etc. Most of these plasters require that the undercoat shall be

of the same type, usually of a coarser grade. Owing to the greater strength of these plasters, they can usually take more sand than the ordinary lime plaster.

Selenitic Cement or Selenitic Lime.—Is made by adding to the limes of the lias formation, which are the best for the purpose, or to the magnesian limestones or any lime possessing hydraulic properties, a small proportion of calcium sulphate in the form of plaster of Paris, mechanically mixed and ground with lime.

The proportion of sulphate the cement contains is from 4 to 7 per cent., the usual proportion being 5. This method is most advantageously applied to the feebly hydraulic limes. When more than $7\frac{1}{2}$ per cent. of sulphate is required to suppress the slaking action, the lime is not suitable for selenitic cement, but may be rendered so by adding a lime that contains more clay.

The sulphate suppresses the slaking action of lime, causing it to set more quickly, and enabling it to be used with a much larger proportion of sand than ordinary lime without loss of strength.

Selenitic lime should not be used in conjunction with gauged stuff for cornices, screeds, etc.

The sand or other ingredients should be always clean and free from loam.

No more selenitic mortar should be gauged than can be used in the same day.

Finely-ground burnt clay (ballast), or cinders, or stone chippings, as a substitute for sand, in whole or in part, can be used with great advantage in every description of work.

Selenitic plastering on walls can be finished in fine weather as two-coat work in 24 hours, while the ceilings can be floated soon after the application of the first coat, and be set in 48 hours, but the time depends upon the state of the atmosphere.

Selenitic cement mortar, with 5 parts of sand, will be found to set harder and more quickly than common mortar with 2 or 3 parts.

Selenitic blue lias is very superior to that prepared from the ordinary grey lime, etc., for all purposes.

Sirapite.—This is a plaster, produced by the Gypsum Mines Limited, of Mountfield, Sussex, and Kingston-on-Soar, Derby, made from gypsum impregnated with petroleum, which latter assists to give it the peculiar characteristics—rapidity of setting, rapidity of drying, hardness greater than ordinary lime plaster—if required it is easily brought to a polished face—does not blister nor blow, does not crack if the timbers do not give, can be used satisfactorily with sawn laths, adheres readily to Fletton bricks ; it is laid on and finished in two coats instead of three. In many districts the cost is less than the ordinary three-coat work.

The rapidity of its setting precludes its use for the running of cornices and moulded bands, its non-porosity when finished to a polished surface causes moisture to condense on the walls, therefore polished surfaces should be prohibited unless a continuous current of warmed air is provided, such as is obtained in mechanically ventilated and heated buildings.

The first coat on walls is composed of one measure of sirapite to three of good clean sand ; for lath work, one of sirapite to one of good clean sand. The second to be of sirapite only, applied as soon as the first coat is sufficiently firm. A small quantity of pure lime putty may be used with the first coat. The sirapite should be mixed in a bunker like ordinary cement, in small quantities, should not be used on permanently damp walls, should be used fresh from the works ; if stored, should be kept in a very dry place ; the work should be thoroughly dry before being decorated. The thickness of the two coats on level work should not exceed $\frac{3}{8}$ inch, which is an advantage for ceilings, compared with the ordinary three-coat plaster work of $\frac{3}{4}$ inch.

Metal lathing is best plastered in three coats—the first to stiffen the lathing, the second to obtain a fair surface. For the first and second coats sirapite and common hair-plaster in the proportion of one to one. The finishing coat to be of neat sirapite.

Sirapite has no corrosive effect upon iron, but tends to preserve it.

Lathing.—Laths are thin strips of oak or fir varying

from 3 feet to 4 ft. 6 in. long and 1 inch wide, nailed at right angles to and on the underside of joists or horizontally on the studs of quarter partitions or walls to form a ground for the pricking-up coat.

The laths are fixed about $\frac{3}{8}$ inch apart, so that the coarse stuff when trowelled on may be squeezed through the interstices and protrude on the back side of the laths, over-

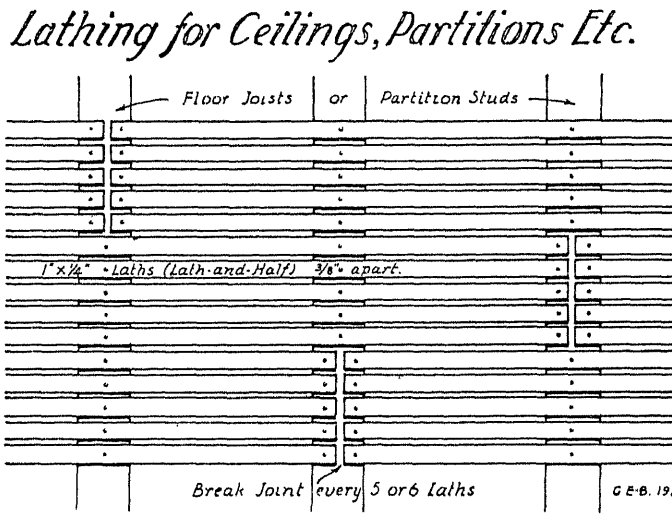


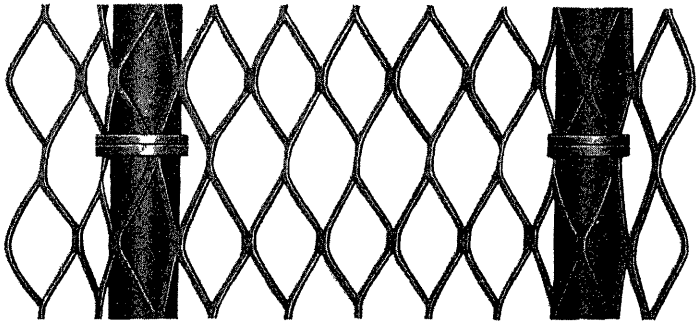
Fig. 6.

lapping the edges, thus forming a key for the pricking-up coat.

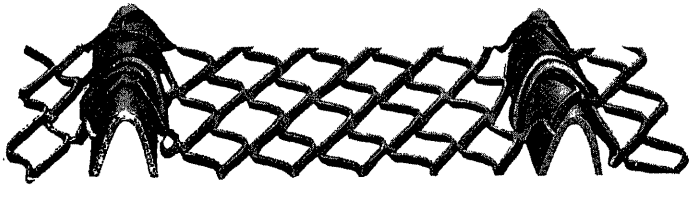
The laths are fixed with iron nails with clout heads; these may be galvanized, wrought, or cast, the first-named being preferred for oak laths. Iron nails are often objected to on account of their rusting and staining the work. Zinc nails are used, but are expensive. The nails vary from $\frac{3}{4}$ inch to $1\frac{1}{4}$ inches in length.

The heading joints of laths must be broken about every foot, and should be butt-jointed and must not lap over each other.

The laths to receive ordinary coarse stuff when prepared



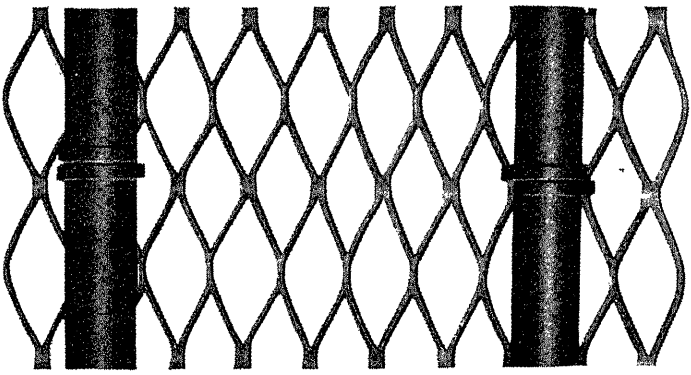
" X " Type, ⅜-inch mesh, " Ribmet."



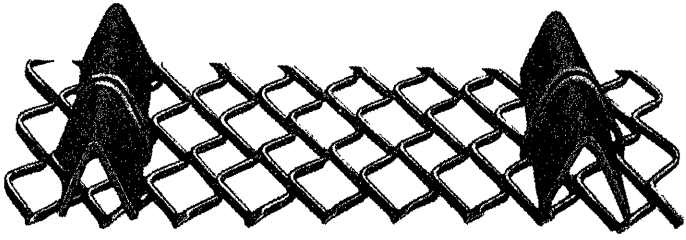
" X " Type, ⅜-inch mesh, " Ribmet." Lathing corrugated over ribs.

Fig. 7.

[*To face p.* 48.



" Z " Type, $\frac{3}{8}$-inch mesh, "Ribmet."



" Z " Type, $\frac{3}{8}$-inch mesh, " Ribmet." Lathing straight under ribs.

Fig. 7A.

should be split or rent from the log, and not cut with the saw, in order to obtain continuous fibres throughout their length. They are made in three thicknesses, and are known as " single laths," " lath and half," and " doubles," the first being $\frac{3}{16}$ inch thick, the second $\frac{1}{4}$ inch thick, the third $\frac{3}{8}$ inch thick. Sawn laths are now commonly used for the less expensive kinds of work (see Fig. 6).

Counterlathing and Brandering.—It is imperative in all lathed work that the key of the plaster should not be interrupted for a greater distance than 2 inches. Where the laths cross timbers of a greater thickness than 2 inches, the key may be obtained as follows :—

First, by counterlathing, that is, nailing laths to the thick timber members in the direction of their length, and over these nailing laths in the ordinary manner ; by this method the key is not interrupted for a distance greater than the width of a lath. Secondly, by brandering, which consists of nailing fillets, from 1" × 1" to $1\frac{1}{4}$" × $1\frac{1}{4}$" at a distance of 1 foot from centre to centre at right angles to the length of the joists, over the whole area to be lathed ; then nail the laths to the fillets in the ordinary manner. This method is especially applicable to cases in which the main timbers are spaced apart a distance exceeding 14 inches, which is the greatest permissible span for a lath.

Metal Laths are now used instead of wood laths to receive the coats of plaster and to resist the action of fire, which they do with satisfactory results for partitions, girders, ceilings, etc. These materials are fixed to iron or steel members by wire or hoop iron passed round them or by nailing to wood cradling pieces. " Ribmet " and " Hy-Rib " (see Figs. 7 and 8) are two of the best-known types, but there are others on the market.

" Ribmet " is composed of The Expanded Metal Company's well-known " BB " expanded metal lathing—either $\frac{1}{4}$ inch mesh, 26 and 24 gauges thick, or $\frac{3}{8}$ inch mesh, 26, 24 and 22 gauges thick, according to which is preferred—with V-shaped ribs attached in the direction of the longway of the sheet. The standard spacings of the ribs are 3, 4 and 5 inches centre to centre, although the material can be

supplied with ribs fixed at other intervals upon special request. Moreover, a heavy or " H " rib, or a light or " L "



" H " Rib. " L " Rib.

(Actual size of Sections.) (*V.* Fig. 7, *ante.*)

rib, can be attached to the lathing, according to the class of work to be done.

" Ribmet " is used for two main purposes—" Z " type for plaster-work, and " X " type for concrete work.

" Z " type " Ribmet " is the only material of its kind in which is provided a continuous lath surface and uniform " key " for plaster throughout ; the lathing passes straight across the opening of the ribs : thus there is no sudden break in the " key " at the ribs, and the possibility of the plaster cracking along them is reduced to a minimum. It is invaluable in the construction of suspended ceilings, false

TABLE OF PROPERTIES

" H " indicates that the rib is of the heavier section—it is made from strip steel 1 inch wide by 19 gauge thick; the overall depth of the " H " weights is $\frac{1}{2}$ inch.
" L " indicates that the rib is of the lighter section—it is made from strip steel $\frac{3}{4}$ inch wide by 24 gauge thick; the overall depth of the " L " weights is $\frac{3}{8}$ inch.
" X " indicates that the lathing is grooved, at the necessary spacings, to receive the ribs.
" Z " indicates that the ribs are attached to the flat sheet of lathing, with the meshwork passing across the opening of the ribs.

Heavy Rib.		Light Rib.		Size of Mesh shortway.	Gauge of Lathing.	Centres of Ribs.
List Reference No.	Approx. weight per yard super.	List Reference No.	Approx. weight per yard super.			
	Lbs.		Lbs.	Inches.		Inches
HX 263 or HZ 263	8	LX 263 or LZ 263	$4\frac{1}{16}$	$\frac{3}{8}$ or $\frac{1}{2}$	26	3
HX 243 or HZ 243	$8\frac{1}{4}$	LX 243 or LZ 243	5		24	3
HX 223 or HZ 223	$9\frac{1}{2}$	LX 223 or LZ 223	$5\frac{5}{8}$	$\frac{5}{8}$	22	3
HX 264 or HZ 264	$6\frac{3}{4}$	LX 264 or LZ 264	$3\frac{13}{16}$	$\frac{3}{8}$ or $\frac{1}{2}$	26	4
HX 244 or HZ 244	$7\frac{1}{4}$	LX 244 or LZ 244	$4\frac{1}{4}$		24	4
HX 224 or HZ 224	8	LX 224 or LZ 224	5	$\frac{5}{8}$	22	4
HX 265 or HZ 265	$5\frac{3}{4}$	LX 265 or LZ 265	$3\frac{1}{2}$	$\frac{3}{8}$ or $\frac{1}{2}$	26	5
HX 245 or HZ 245	$6\frac{1}{2}$	LX 245 or LZ 245	4		24	5
HX 225 or HZ 225	$7\frac{1}{4}$	LX 225 or LZ 225	$4\frac{13}{16}$	$\frac{5}{8}$	22	5

Dipped once in Asphaltum paint unless ordered otherwise.

arches, mock beams, solid and hollow walls and partitions, and for many other classes of plaster construction, where the bearers are too far apart for plain sheet lathing.

The " HZ " weights may be used horizontally up to 3 feet spacings and up to 4 feet vertically; the " LZ " weights are suitable for spacings up to 2 ft. 6 in. and 3 ft. 3 in. respectively. " Ribmet " having 22 gauge lathing is recommended for special ornamental and exterior work.

" X " type " Ribmet " also offers many advantages as a combined permanent centering and reinforcement for concrete flooring, roofing, etc.; it eliminates the cost of preparing, erecting and removing temporary sheet centering which is particularly expensive for curved work.

In the case of " X " type " Ribmet " the lathing is corrugated over the ribs; thus the greater part of the weight of the concrete is taken by the ribs; the mesh is small enough to retain the concrete while it is being laid, and it forms a " key " for a plaster soffit.

The " HX " weights, being the stronger, are applicable especially in concrete work, although " LX " weights also are suitable for short spans.

" Ribmet " can be bent in sheet form to special shapes for various purposes; and it can be curved or cambered to practically any radius required in building and engineering work generally.

" *Hy-Rib.*"—One of the most widely used reinforcements for concrete work is " Hy-Rib " (a product of The Trussed Concrete Steel Co. Ltd., London), which over a period of many years has established its efficiency for a very wide range of uses.

" Hy-Rib " is a steel lathing stiffened by rigid high ribs, and is manufactured from single sheets of British steel, the lathing and the ribs forming a complete unit (Fig. 8). It is supplied in standard sized sheets of varying gauges for different usages; 10½ inches wide from centre to centre of the outer ribs, and in lengths from 6 to 12 feet. The outer ribs are made to interlock so that sheets can be built up side by side to any desired width, and is obtainable in flat sheets for general work, or curved for arched floor work.

One of the greatest advantages of " Hy-Rib " is that it is a combined centering and reinforcement, and its use completely eliminates the need for close-board shuttering ; thereby reducing time of construction and costs by cutting out a complete labour operation. The rigid ribs enable concrete slabs to be carried in the same manner as timber boarding, whilst the unique design of the steel mesh retains the wet concrete and acts similarly to the boarding which it replaces. The mesh grips the concrete into a sound

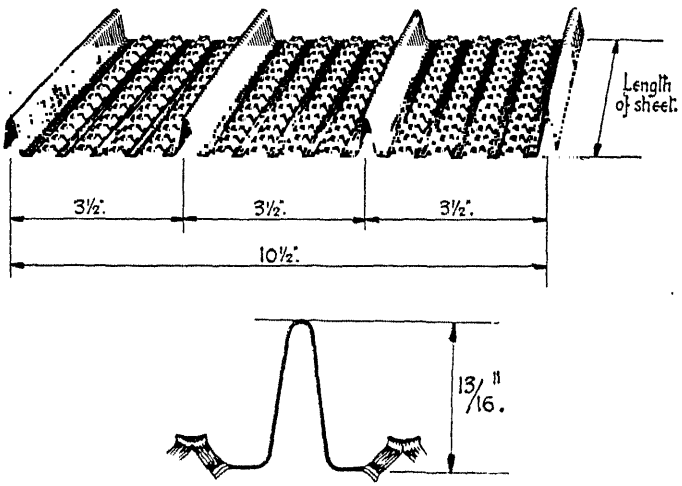


Fig. 8.—" Hy-Rib " Steel Lathing.

anchorage which evenly distributes the stresses in a slab and ensures a rigid and strong floor or roof.

With " Hy-Rib " as a basis, it is possible to construct the thinnest slab allowable in reinforced concrete design, with a resultant saving in dead weight on the structural framework.

The uses for " Hy-Rib " are innumerable, and in the course of a day's work the builder will encounter an infinite variety of jobs the construction of which will be improved, and the cost lessened, by its use. Some of the more impor-

tant and general applications being for walls, floors, partitions, roofs, ceilings, balcony steppings. In its curved form it will be found invaluable for arched floors, tanks, conduits, culverts, silos, reservoirs and similar structures.

For interior walls, ceilings, etc., where plastering is necessary, the " key-mesh " will grip the plaster in such a tenacious manner that falling is practically impossible, even where extreme vibration occurs. The plasterer's labour is simplified and an excellent result is obtained.

In addition to the durable and the hygienic properties of " Hy-Rib " structures, there is the ever-important quality of fire-resistance which makes this material an almost indispensable asset in modern buildings. " Hy-Rib " can be recommended strongly for such buildings as cinemas, theatres, domestic houses, in fact, for all buildings designed and used to house human life.

Sand.—All sand to be used for plastering should be clear from clay or loam, for the presence of a very small quantity of either of the latter foreign matter would very much weaken the compound and reduce its setting powers. Sand is rarely found in the required condition, hence it nearly always requires to be washed, a small quantity being placed in a sieve and turned to and fro in a tub of water. For the first coat the sand may be washed through a sieve having about sixteen meshes to the square inch ; for fining and for finishing mouldings, a sieve having from 256 to 576 meshes to the square inch, according to the degree of finish required. For the finishing off of Portland cement work, a mixture of ordinary sand, and either silver sand or stone dust with the Portland cement is used. Pit sand is sharper than river sand, and is best for Portland cement.

River sand, which is fine and of a light colour, is preferred for internal plasterer's work, the matrix of which is lime or Keene's cement.

Sand is now washed by machinery much more efficiently than by hand.

The sand-washing plant is usually erected at the pits, and the sand is supplied by the merchants in the washed form.

Plastering Processes.—The process of covering walls with plaster is usually accomplished in either two or three coats. If the first coat be applied to brick, concrete or stone walls, it is termed " rendering," if to a lathed surface it is termed " the pricking-up coat." In good quality work three coats are employed, *i.e.*, the rendering, the floating, and the setting coat. In brick or stone work the mortar joints should either be raked or left protruding to form a key. The surfaces should be rough and absorptive, to help the adhesion. If the surfaces are smooth they should be hacked before the application of the plaster and well brushed to remove all dust, and wetted. Wall surfaces of large hard stones which afford very little mechanical key are usually strapped, that is, 2″ × 1″ wood fillets or strips are fixed to plugs in the wall. The surfaces of the straps are carefully fixed in a true plane surface. These are lathed and a pricking-up coat is applied to the laths. In lathing, the whole adhesion of the plaster depends upon the mechanical key.

The object of the rendering or the pricking-up coat is to form a ground for the subsequent coats. Before this coat has set it is scored with some pointed laths known as a scratch (see Figs. 13–41) to form a key for the floating coat. The object of the floating coat is to form a true surface. For this purpose wood rules are fixed at all external angles, their working edges being kept perfectly vertical. If the width of the surface between the rules is under 10 feet the plaster is trowelled on to the surface and all superfluous material is struck off with a long straightedge known as a Derby float. The true surface thus formed is scratched with a hard broom to form a key for the setting coat. For surfaces between internal angles, vertical plaster screeds are made a few inches from the angles. These are usually formed by driving nails, which are left projecting, at the top and bottom of the screeds, and are carefully plumbed. A strip of plaster, 2 to 3 inches wide, is then applied between the projecting nail heads, and is struck off truly with a straightedge. When the screeds are set the nails are withdrawn. Instead of nails, dabs of plaster are sometimes employed.

If the surfaces are wider than 10 feet, intermediate

screeds are formed, being kept true between the end screeds by lines and rules. The areas between the screeds are then plastered, and superfluous material between the screeds is struck off with a Derby float. The process of internal screeds applies to surfaces wider than 10 feet between external angles. The narrow wood grounds used for fixing wood skirtings, dado and picture rails and door architraves, where these are employed, are used as screeds by the plasterer for the floating coat. The setting coat, which should never require to be more than $\frac{1}{8}$ inch thickness, is applied with the trowel and finished with the hand float.

Two-coat work is used on the cheaper kinds of work. Here the pricking-up or rendering coat is applied with the closest approximation to accuracy, advantage being taken of all external angles to fix rules, to ensure their verticality. Although quite good surfaces can be obtained by skilful plasterers, the surfaces are never as true as with the three-coat work.

Internal Plastering.—For internal plastered surfaces the first and second coats are known as coarse stuff. It consists usually of 1 part by volume of lime to 2 parts of clean sharp sand and long well-beaten ox-hair in the proportion of 9 lbs. of hair to 1 yard of mortar. The usual method of preparation is as follows. A ring or basin of the sand is formed on the ground about 2 yards in diameter. At one point in the circle is a tub with a rough frame supporting a fine sieve. Into the tub, which is filled with clean water, freshly burnt lump quicklime is emptied. This is vigorously stirred till the lumps are broken up. The mixture is then baled out with a pail and passed through the sieve. Anything not readily passing through the sieve is rejected and thrown on one side. This hydrated lime, when passed through the sieve, is known as putty. There is a considerable amount of water in excess. When the lime has cooled, the ox-hair, which has been previously well beaten to separate the individual hairs, is evenly distributed over the surface of the putty The sand, putty, and hair are then mixed and turned over and worked with a larry till the whole is of a uniform consistency. It should then be heaped and kept for a few days before using to ensure

that the lime is thoroughly slaked. Any unslaked particles worked into the surfaces will eventually slake and blow, resulting in a pitted surface. The commercial hydrated lime previously described is now rapidly displacing the lime putty, owing to its greater reliability as regards slaking and the elimination of the running process. Being in the form of a fine powder, it is more easily mixed with the sand and hair, and only requires the addition of the necessary water. Its general advantages have been described on p. 5.

Portland cement coarse stuff in the proportion of 1 Portland cement to 3 of sand, makes an excellent ground coat on brickwork, but on lathing the water required for the setting and the hardening process to develop is liable to dry out before those processes are complete.

Gauged Stuff.—For ceilings, gauged stuff is usually employed, especially with lathed surfaces to accelerate the setting.

Gauged stuff consists of the coarse stuff preferably prepared with hydralime, with a proportion of about 1 to 6 of coarse plaster. With ordinary plaster, however, the mixture sets very rapidly. It must therefore be mixed in small quantities, being gauged usually on the banker and used immediately.

For this work it is usual now to employ a retarded hemihydrate plaster. With this the set is delayed for about 2 hours, thus giving ample time for the manipulation. There are many such retarded hemihydrates on the market, such as Gothite, Murite, Carlisle, Hardwall, Thistle, etc., suitable for gauged work. These mixtures not only set quicker than ordinary coarse stuff, but are much stronger when set. Gauged stuff is also made with a proportion of Sirapite or Portland cement.

The following proportions give excellent results for rendering or pricking-up coats :—

Hydralime	2 parts
Sand	6 „
Hemihydrate plaster or Sirapite ...	1 part

The hemihydrate may be replaced with about 10 per cent. of Portland cement.

Setting Coat.—The setting coat is usually made from plasterer's putty mixed with about 2 parts of clean washed sand. The putty for this purpose is usually run in the manner previously described, but into a specially constructed bin, where it is kept for some time before gauging with the sand. The thickness of this coat should never exceed $\frac{1}{8}$ inch. Being very thin, the lime readily takes up CO_2 and thus sets very hard. It is usually trowelled to a smooth even surface and finished with a hand float. The surface is kept constantly moist by drawing a stock brush over the surface and floating with a circular motion. This hardens and consolidates the surface. The process is known as " scouring." Setting coats contain no hair, but may be gauged with any of the hemihydrates employed in the ground coats.

External Work.—The covering of external surfaces is carried out in a manner similar to internal work. As the covering is usually applied to brick surfaces, two-coat work only is employed. The first coat is not only a rendering coat, but also a floating coat, as the screeds can be formed direct on to the brick surfaces.

Stucco.—External plastering is usually known as stucco. It has been used from the earliest known times as a protection from the weather and as a medium for decoration.

Up to the end of the seventeenth century, lime mortar was employed. For good work, the greatest care was taken in the selection and treatment of the lime. It was thoroughly slaked and kept in covered pits for considerable periods before being used. Clean sand, marble dust and other such media, graded and selected with care, were mixed with the lime. Hair or fibre was mixed with the first coat to act as a binder. For the final coat, lime and sand only were employed. Various finishes were given to the material. Sometimes it was trowelled up to an exceedingly smooth, hard and almost polished surface or, again, it was modelled into ornament and various designs worked on it. Sometimes repetitive ornament was stamped on to the plaster while still soft.* Textural finishes to represent

* See Millar's "Plastering: Plain and Decorative," and Jourdain's "English Decorative Plasterwork of the Renaissance."

stone surfaces were worked with a felt-covered hand float. A finish known as rough cast was formed by mixing fine pebbles in with the lime, the material being applied with the trowel, or a variant known as pebble-dash was given by dashing handfuls of small pebbles on to the freshly applied finishing coat.

Portland Stucco.—Lime is now almost superseded by Portland cement as the matrix for stucco work, owing to its greater strength and uniformity, and also for the greater speed with which the work can be executed. When the necessary care is taken properly to grade the sand, the resulting surface is practically waterproof. The objection that held up the use of cement for long was the liability of surface crazing due to the expansion and subsequent contraction of the cement. This on wall surfaces was largely accounted for by the difficulty of maintaining the necessary water in the compound to complete the setting action. In addition to the mechanical key afforded by the projecting mortar joints or the raked-out joints and hacked surfaces, the surface should be thoroughly brushed and wetted, to prevent the water used for mixing from being absorbed from the cement. All subsequent coats should be allowed time to set properly, be scratched to provide a good mechanical key, and wetted to ensure proper adhesion. Even with ordinary cement, crazing is still liable to occur, due to the suction of the ground or undercoats. To avoid this, a water-repellent cement should be employed in the rendering coat. This does not absorb any of the water from the setting coat. In this case the suction will be eliminated, but the adhesion of the two coats will also be reduced, the bond between the two being almost entirely mechanical. The general proportion for both the rendering and the setting coats is about 1 to 3 of Portland cement and clean gritty sand. Ordinary Portland cement has an unattractive colour, but this can be rendered pleasing by the addition of pigments or by employing sands of various tints, or by aggregates made of crushed limestone marbles or granite. The Associated Portland Cements Ltd. have marketed material for the finishing coat termed Cullacrete, a material composed of white

ferrocrete known as Snowcrete, mixed with a wide range of pigments and selected sands or aggregates properly graded, the whole being uniformly proportioned to give the best results both for strength and uniformity of tint.

Where pigments are mixed with the cement the amount should never exceed 15 per cent., or the strength and damp-resisting properties of the material will be affected.

Rendering coats are liable to injury by excesses of temperature or by drying winds. Frost will suspend the setting action, and a high temperature will cause a rapid evaporation of the water necessary for setting. It is therefore advisable to cover the surfaces with some fabric to protect them from frost or wind, or from drying too rapidly during periods of high dry temperatures.

Lathed Surfaces.—Where the ground for rendering is lathed it is preferable to employ steel lathing instead of wood. The latter, owing to its frail and absorptive nature, absorbs some of the water of setting, and thus affords only a mechanical key to the first coat. Expanded metal, or "Hy-rib" are suitable materials, as both provide an excellent key for the rendering. They are non-absorptive, and there is a measure of adhesion.

For the first coat for lath work, a mixture of lime putty with 10 per cent. of Portland cement and 2 to 3 parts of sand with the usual proportion of hair is recommended. This mixture works fatter and is more adhesive than the mixture of cement only. Wherever rendering is applied to timber surfaces, the latter should be covered with expanded metal rigidly fixed with sufficient space behind to form a good key.

Fibrous Plaster.—For internal work fibrous plaster slabs are often substituted for the pricking-up and floating coats. These are usually made in sheets convenient for transport and handling, about 3′ 0″ × 1′ 6″. They can be made, however, in much larger sheets. Fibrous plaster slabs are usually made on a bench, the top of which is covered in zinc or plaster screeded into a plain surface. About the edges is a frame made of wood about 3″ × 1″ and screwed to the bench. Placed in this is a secondary

frame made of loose pieces secured with buttons as shown in Fig. 9. Sufficient plaster is gauged as a thin slip, poured into the mould and spread evenly. On to this, coarse open-web canvas is spread and pressed with the trowel on to the canvas. Laths are then placed along the four edges, and the edges of the canvas folded over them. More plaster slip is then poured on to cover the canvas, also the reinforcing laths. When set the pieces of the loose frame are removed, the slabs are carefully raised from the bed and stacked on edge in a rack to harden. The edges of the loose wood strips are bevelled. The slabs are fixed to the wood studs or joists with 1½-inch galvanized flat-headed nails driven at about 6-inch centres. The heading joints of the slabs should be broken. The joints are stopped with plaster gauged with size water.

The slabs are sometimes made solid. After the laths have been bedded, plaster gauged with size water is poured over the top and struck level with the side strips. The surface is scratched with a metal comb or drag to form a key for the finishing coat.

Large cornices are usually run in lengths of about 6 feet to 8 feet. A reverse of the profile is run in plaster. This is coated with shellac or varnish, and is greased with oil or a tallow rag. The plaster slip is then poured on. If the surface is large, the set is delayed by gauging the plaster with size water. The canvas reinforcement is then laid on and pressed into the plaster. This is stiffened with wood fillets, or round iron up to about ¼ inch diameter. A backing slip is then trowelled on to the back of the canvas, enveloping the wood or iron rods. The cornice is fixed in sections, being nailed or screwed to prepared wood backings. The joints are carefully stopped with plaster. Ornament is usually modelled first in clay. Over this a plaster slip is poured. This is thickened up to a thickness of about 2 inches. These moulds usually have to be made in two or more pieces, so that the mould can be released from the final cast. In this case, joggles must be worked on the joints of the various pieces of the mould. Then when all the pieces of the mould are together, another plaster enveloping box is cast around the piece mould. The mould is made tapering so that it can easily be released

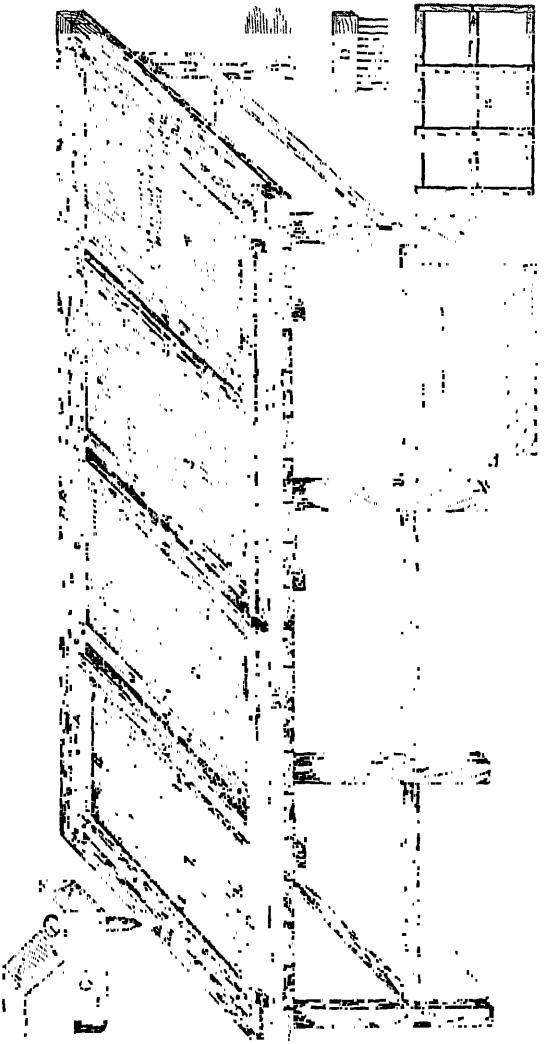


Fig. 9.—Bench for Casting Fibrous Slabs.

from the enclosing plaster box. To cast the ornament, the clay model is first cleaned out of the piece mould. The latter is coated with shellac, to prevent it adhering to the cast. The surface should be rubbed with a tallow pad to facilitate further the separation of the cast from the mould. A plaster slip is poured in to the mould, care being taken that every portion of the surface is covered. This slip is backed up with plaster to a thickness of approximately 1 inch. The plaster sets in about 5 minutes, but it should be left for a time to harden before it is removed from the mould. The mould is then taken out of its plaster box, and the pieces of the mould carefully removed from the cast. The mould is then cleaned, greased and put together again preparatory to the next cast. Where the ornament has many undercut surfaces considerable ingenuity is required to construct the pieces of the mould so that they can be released from the cast without injury to the latter.

Where there is much undercutting on the model, gelatine is employed which, owing to its elasticity, can be drawn from the undercuts, thus avoiding the manufacture of complicated piece moulds. Gelatine, which is a superior kind of glue, requires considerable skill in its manipulation (for further reference see Millar's " Plastering "). India rubber is now being employed for moulds for reproducing undercut work. This material, owing to its greater strength and tenacity, may supplant gelatine.

Architectural ornament is now being largely carried out in Portland cement. Features, such as balusters, etc., that have to be repeated many times, are cast in moulds. Plaster moulds are unsuitable for cement moulding; metal moulds are therefore employed. In these, the cement can be rammed without injury.

Window sills and steps may be cast *in situ* or precast. Where there is much repetition, the precast method is employed. The members are usually cast in wood moulds, the surfaces of which should be planed smooth. When casting, a fine mixture of cement and sand 1 to 2, should be trowelled over the faces of the mould to a thickness of about 1 inch. The core can then be of a coarser mix. A reinforcement of expanded metal is usually embedded in

the core. White or coloured cement can be employed for the seen faces if desired.

Running Mouldings.—Plain moulded work is run with a profile of the moulding cut in reverse in zinc. This is

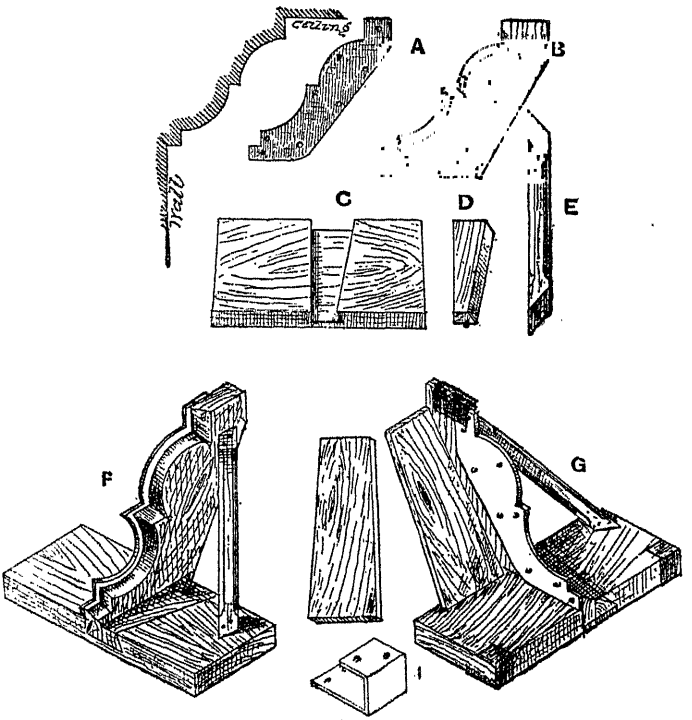


Fig. 10.—Horses for Running Moulds.

mounted on a stock (see Fig. 10), the latter is roughly cut to the profile, but is kept about ½ inch back from the zinc edge. This stock is mounted in a slipper, a piece of wood in length about twice the projection of the moulding. The stock is housed into the slipper and kept at right angles to it by a stay-bar screwed at one end to the stock and at

Fig. 11.—Bracketed and Lathed.

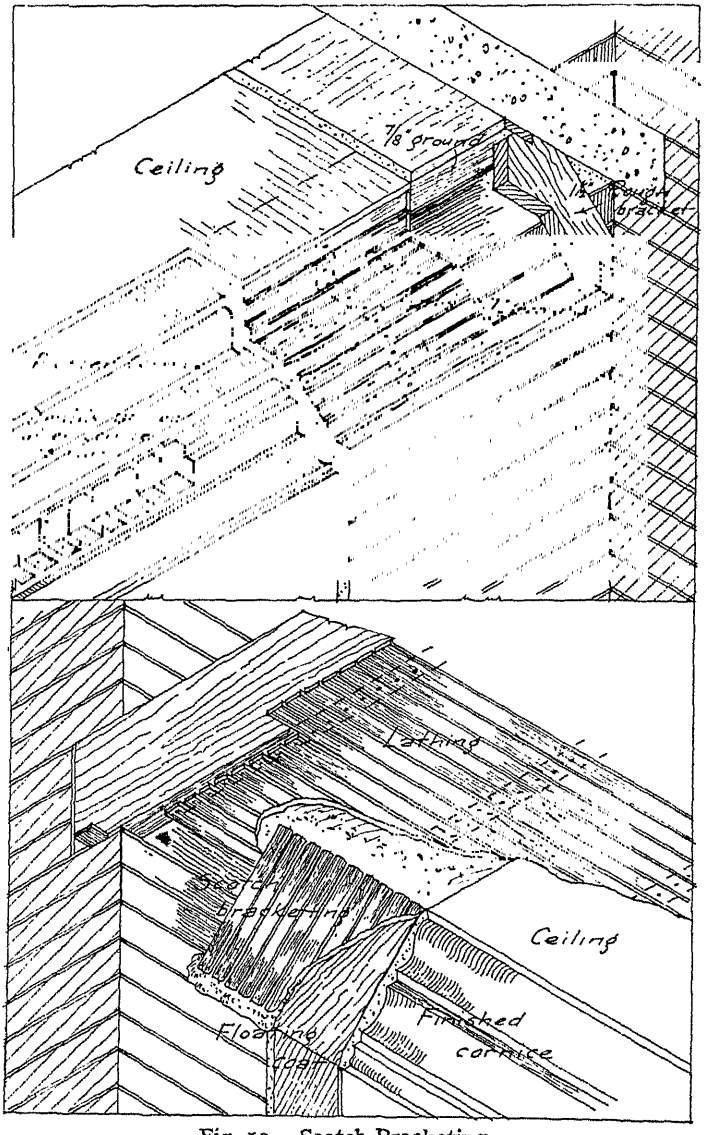


Fig. 12.—Scotch Bracketing.

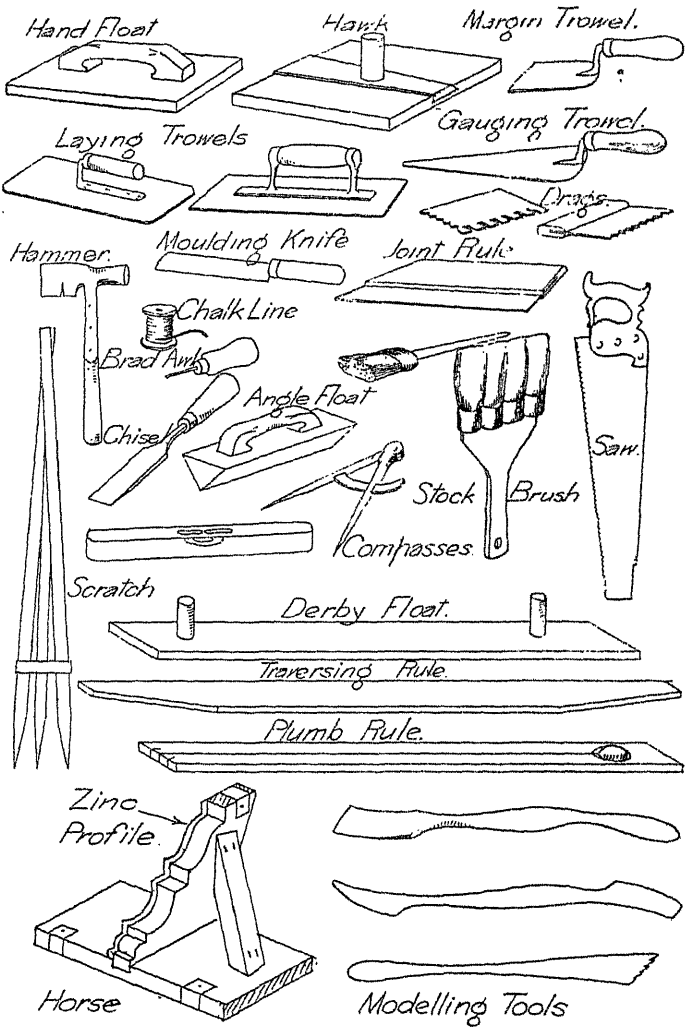
Figs. 11 and 12.

the other end to the slipper. This stay also forms a handle for the plasterer to manipulate the slipper. Two rules are required about 2″ × ¾″, one fixed on the wall and the other on the ceiling. The edge of the slipper and the nib of the stock are run upon these. The running edge of the slipper has two pieces of zinc about 1 inch wide fixed near each end to take up the wear. There should be a similar piece on the nib of the stock.

Where the cornices are not more than 6 inches projection or where they are coved so that there is no great thickness of plaster, they are made solid, but where the projection is greater than 6 inches the cornices are bracketed out, and lathed, or if the profile is convenient Scotch bracketing is employed (see Fig. 12). This consists of two screeds of plaster at the requisite distance from the wall and ceiling, and laths pressed into the screeds while they are still soft. The ground surface of coarse plaster is trowelled on to the laths, and when sufficiently set the setting coat is applied. The plaster is laid on and the profile is run along the rules, the superfluous material scraped away, the hollow spaces filled up, and the scraping process repeated till the moulding is complete. Where there are mitres, either internal or external, the profile is worked up as near to the mitre as possible. Then the mitre is completed with steel joint rules. Running ornament, such as egg and tongue dentils, etc., is precast and applied when the running of the cornice is completed.

Skirtings and dados are run in a similar manner, using Keene's or Portland cement where there is likely to be any hard wear or exposure.

Plasterers' Tools.—Figs. 13–41 illustrate types of the ordinary tools of the plasterer: the hawk, for holding material preparatory to depositing upon the surfaces; laying trowels, for laying on the coarse stuff; hand float, used for laying on setting coats; gauging trowel, for mixing gauged stuff; margin trowel, for angles and small spaces where a larger tool would be inconvenient; scratch, used for forming the key in the first coat; angle float, for working up right angles; drags, used to form a key for the next coat, and also for straightening surfaces; joint


Hand Float
Hawk
Margin Trowel.
Laying Trowels
Gauging Trowel.
Drags.
Hammer.
Moulding Knife
Joint Rule
Chalk Line
Brad Awl
Angle Float
Chisel
Saw.
Stock Brush
Compasses.
Scratch
Derby Float.
Traversing Rule.
Plumb Rule.
Zinc Profile.
Horse
Modelling Tools

Figs. 13—41.

rule, for working up angles of cornices; hammer, for lathing and general work; chalk line, compasses, level, and plumb rule, for setting out and testing work; saw, for cutting rules and for general work; moulding knife used in making good to work; traversing rule, for forming plaster screeds; Derby float, used for floating material between the screeds; stock brushes, for wetting plane surfaces and moulding during their formation; horse, for running mouldings, a tool specially made for each varying section. There are varieties of each of the previously mentioned tools, also others used for special work, such as modelling, three specimens of which are shown.

CHAPTER III

ASPHALT

ASPHALT as a waterproofing compound was introduced into England about 1837 by Seyssel. It had been used in France for about 100 years previously. Its adoption was slow because its nature and properties were not fully understood. As its merits became more generally known the demand increased; but its peculiarities were not properly understood and the results obtained were not always satisfactory.

In recent years this has led to the formation of the Natural Asphalt and Mine Owners Council: a body consisting of the principal mine owners and manufacturers in this country. The Council has carried out a considerable amount of research and experiment to ascertain the composition and nature of the material in order to fix standards of quality, the best methods of manipulation, and to ensure the best results for any specific purpose.

Manufacturers affiliated to the Council must provide approved machinery in order to ensure that their products satisfy the required standards. As with all natural products, natural rock asphalt is variable in composition, even in the same mine. Therefore arrangements have been made that the supply of the raw material from many well-known mines shall be available to all members of the association. Thus a product of uniform standard of excellence for any constructional purpose can be guaranteed.

Bitumen, the basic constituent of asphalt, is a compound of hydrocarbons derived from petroleum—which in turn has probably been formed from organic matter of animal and vegetable origin deposited during the Carboniferous Period.

Mastic Asphalt, the material used in construction, is a blend of three materials: (1) Natural Rock Asphalt, (2) Trinidad Lake Asphalt, (3) Asphalt Flux Oil.

Natural Rock Asphalt is a rock impregnated with bitumen. The rocks, usually limestones, are finely granulated and intimately mixed with the bitumen. The chief sources of supply are: The Val de Travers, Neuchatel, Switzerland; The Seyssel region, Bassin de Seyssel, France; St. Jean de Maruéjols, Garde, France; The Limmer, near Hanover, Germany; and Sicily. The rock is crushed and ground to a fine powder, ready for mixing with the other constituents.

Lake Asphalt is obtained from the asphalt lake in the Island of Trinidad and is very rich in bitumen. The mineral matter is in a fine state of subdivision and the whole is uniform in substance. It is excavated from the surface and conveyed to the refining plant, where any impurities are removed, and is then run into barrels for export.

Asphaltic Flux Oil is derived from petroleum by a process of distillation. Its function is to soften the lake asphalt and to act as a flux in the process of blending.

In the process of manufacture the constituents are melted in mechanically agitated mixers to form a homogeneous compound. This, when ready, is cast into blocks of about 56 lbs. in weight, ready for transport. During the process of manufacture great care is taken to control the temperature, which must never exceed 400° Fahr. The resulting compound is known as Mastic Asphalt.

Asphalt is a rich brown colour, is sanitary, damp resisting, non-absorbent, non-inflammable, hard wearing, and slightly plastic. When preparing the material for use, the blocks are broken, placed in boilers brought to the site, melted, and the molten mass constantly stirred with iron rods. Care is taken to prevent overheating. When ready the asphalt is conveyed in pails to the spreader, who applies it to the surfaces to be covered with floats.

The advantage of asphalt over all other materials for excluding moisture is that when completed it is jointless; hence no dampness can enter by capillarity. Asphalt is employed in building wherever it is desired to exclude dampness.

Tanking.—In basements constructed below the ground

level in saturated soils, a tank is formed enclosing the structure and covering the floor (Fig. 135). In new buildings it is usual to apply the asphalt to the outside of the brickwork (Fig. 166 E.B.). The joints of the brickwork are raked out to form a key. It is applied in three layers to a minimum finished thickness of $\frac{3}{4}$ inch for vertical work and $1\frac{1}{8}$ inch for horizontal work. There should be a double asphalt fillet at internal angles. The asphalt on the floor surface, laid on the oversite concrete, is made continuous with the external wall covering. Owing to the low adhesive value of the asphalt, it is necessary to cover the horizontal surface with a loading slab of sufficient weight to resist the upward pressure of the water, or a thin reinforced slab may be used (Fig. 166 E.B.).

In existing buildings, basements may be waterproofed by applying the asphalt in a similar manner. The vertical asphalt is applied inside the wall and an inner skin wall built to hold the asphalt in position (Fig. 166 E.B.).

Stanchion bases, whether bloom plates or grillages, which are usually placed some distance below the floor level, should first have their pits lined with concrete (Fig. 133). The bottom and sides are then tanked with asphalt; the upper edges of the tank are jointed to the floor asphalt, and a concrete layer is then bedded on the floor of the pit to receive the base of the pillar. When the latter has been placed, levelled, and grouted in position, the whole is concreted to the level of the floor concrete. Thus the steelwork is protected from any corrosive action due to moisture (see Fig. 133). Figs. 128, 129 and 135 show methods of waterproofing area retaining walls constructed in front of buildings. The asphalt should be spread on the vertical skin walls in three layers—never less than in two layers.

Roofs.—The usual form of roof to be covered with asphalt is the flat roof. This may be constructed either with wood or concrete; may have parapet walls or an eaves finish. The usual thickness of asphalt on roofs is $\frac{3}{4}$ inch applied in two $\frac{3}{8}$ inch layers (see Figs. 1171–1176 E.B.).

With boarded flats an underlay of felt should be provided to avoid any defects arising through movements in

the woodwork. The covering is sometimes reinforced (Fig. 1173 E.B.), but this is unnecessary on approximately flat surfaces. When reinforcement is used, expanded metal gives the best results. Where there is a sloping or vertical skirting, the angle between the flat and the skirting should be reinforced with expanded metal. This applies also in valley and box gutters where the surfaces are sloping or vertical and are formed of wood. Where the skirting is formed against brickwork, reinforcement is unnecessary.

On concrete surfaces the flat is rendered to form the required falls, minimum 1½ inches in 10 feet, and the asphalt is applied direct. The skirting against parapet walls is made a depth of at least 6 inches and is tucked into a raglet for a depth of 1 inch. The thickness of the joint in brickwork is insufficient for this turn in, and the raglet is widened by cutting away the top edge of the bricks on the lower side in order to give a thickness to the tucked-in asphalt of at least 1 inch. The asphalt may be spread over the whole thickness of the wall at this part to form a horizontal damp-proof course. All internal angles between vertical and horizontal surfaces should be strengthened by forming a triangular fillet at that part.

Where the roof is formed with an eaves finish it is necessary to fix a lead or copper flashing at the actual edge, projecting for at least 3 inches on the flat surface, the inner edge of the metal being welted. The asphalt is then laid over the horizontal surface of the flashing, tucked slightly under the edge of the welt, and finished at or near the edge of the eaves (Fig. 1172 E.B.). The outer portion of the metal flashing is dressed down the wall face for at least 6 inches, or into an eaves gutter, or over the slated or tiled sloping surface below the flat.

Advantages of an asphalt roof are (*a*) it has a seamless surface, (*b*) it is tough, (*c*) it is hard wearing compared with metallic coverings. Hence it can be subjected to a considerable amount of traffic without injury.

Damp-proofing.—Figs. 153–162 E.B. illustrate examples of horizontal damp-proof courses, for which purpose mastic

asphalt is supreme, both at the base and summit of walls.

Above ground level it may be applied in one ½-inch coat, kept back ¾ inch from the face of brickwork to allow for pointing. In order to prevent any possibility of the material squeezing under load, the inclusion of a moderate proportion of grit of a suitable grading may with advantage be incorporated.

Below ground level, other than in *Tanking*, both horizontal and vertical work should be in two layers, the first of ¾ inch thickness and the second not less than ½ inch thick. All the internal angles must have an asphalt fillet.

Artificial Asphalts.—Artificial asphalts known as British are formed of an admixture of coal tar, pitch, chalk, quicklime, sand or sawdust, and ground iron slag, heated and laid in a semi-fluid state. These are, as already stated, considerably inferior to the natural asphalts.

Coal Tar Pitch.—Coal tar is a bye-product in the manufacture of gas from coal. The products obtained by the fractional distillation of coal tar, which are, relatively to pitch, of considerable commercial value, include: light oil, up to 170°; middle oil, 170° to 230°; heavy oil, 230° to 270°, used as creosote; anthracene oil, above 270° Cent., and the residuum is pitch. Coal tar pitch, at times thinned down with some of the heavier tar oils and sometimes called prepared tar, is heated and mixed with broken limestone or Kentish ragstone, forming a kind of concrete, and is known as tar paving; the limestone is screened in sizes from 1¾ inches to ⅜ inch, the gauge varying according to the thickness of the layer in which it is to be used. The paving is laid in two coats. The thickness of bottom layers varies from 3 to 1½ inches, and top layers from 1 to ½ inch, the total thickness for paths being 2, 2½ and 3 inches for light, moderate and heavy traffic respectively.

Solid slag run from iron blast furnaces, or approved granite such as Guernsey, broken to a 2-inch gauge and saturated with coal tar pitch, is used successfully as the surface covering in modern roads subjected to traffic of medium weight and of rapid speed, and is efficient in overcoming the dust and noise nuisances from such traffic.

BRITISH STANDARD SPECIFICATION FOR MASTIC ASPHALT FOR ROOFING (No. 988-1941)

FOREWORD

The immediate issue of this Specification for Mastic Asphalt for Roofing, the work for which was started some years ago, was undertaken as a result of representations that there was a need for specifications for this class of materials.

In issuing this publication the Committee desires to emphasise the fact that these specifications are not in themselves war-time formulæ. The Committee are advised that mastic asphalt as defined herein has been widely used for many years past under guarantees of performance. Users are therefore reminded that the specifications purport only to contain technical provisions, but do not purport to include all the necessary provisions of a contract.

The Committee also desires to emphasize that the types and grades of mastic asphalt defined herein are based on the experience and technical practice of certain branches of the industry. They have been drawn up to employ ingredients normally used in peace-time and these are still available.

The Specification does not at present include types of mastic asphalt which embody those ingredients which are not available at this time, such as natural asphalt rock, flux, oil, etc., the importation of which into this country is at present impossible.

A final complete Specification for Mastic Asphalt or Roofing will be issued in due course and will cover and embody all the appropriate types and grades, including those referred to above.

It has been the endeavour of the Committee to make the Specification as useful as possible by the inclusion of recommendations for the method of application.

N.B. Attention is drawn to the fact that unsuitable camouflage paint may cause considerable damage to asphalt roofs. (See Part 6, p. 79.)

SPECIFICATION

PART 1. SCOPE

1. *Scope.*—This Specification provides for the following types of material, designated as Type A :

Mastic asphalt for roofing composed of limestone aggregate incorporated with either :

(*a*) Asphaltic bitumen.

or

(*b*) Equal proportions of asphaltic bitumen and refined Lake asphalt.

NOTE.—The proportion of refined Lake asphalt may be varied by agreement between the architect or engineer and the asphalt contractor, but shall not exceed 50 per cent. of the total asphaltic cement.

Specifications for other types of materials are under consideration.

PART 2. DEFINITIONS

2. *Definitions.*—For the purposes of this Specification the following definitions shall apply :—

(*a*) *Asphaltic Bitumen.*—A semi-solid product from the distillation of asphaltic-base petroleum, consisting essentially of hydrocarbons and substantially soluble in carbon disulphide.

(*b*) *Lake Asphalt.*—An asphalt which, as found in nature, is in a condition of flow or fluidity.

(*c*) *Limestone.*—Limestone is here interpreted in a restricted geological sense, and denotes a naturally occurring consolidated stratified calcareous rock of aqueous origin, containing not less than 90 per cent. of calcium carbonate.

(*d*) *Grit.*—Any silicious material obtained from natural deposits either directly or by screening, crushing or other recognised mechanical process.

PART 3. GENERAL CLAUSES

3. *Gauges of Materials and Dimensions of Sieves.*—

When material is herein specified by gauge, such material shall be tested according to B.S. 812 Sampling and Testing of Mineral Aggregates, Sands and Fillers, Part 1.

4. *Asphaltic Cement.*—The asphaltic cement as selected shall comply with the appropriate specification given in Table I.

TABLE I.—*Properties of Asphaltic Cement*

Property.	(a)	(b)
Softening point (ring and ball).	122° to 158° Fahr. (50° to 70° Cent.)	122° to 158° Fahr. (50° to 70° Cent.)
Penetration at 77° Fahr. (25° Cent.) . . .	20 to 45	20 to 45
Ductility 77° Fahr. (25° Cent.)	Min. 40 cm.	Min. 30 cm.
Solubility in CS_2 (per cent.)	99·5	75 to 79
Mineral matter (ash) (per cent.)	Max. 0·5	16 to 19
Loss on heating for 5 hours at 325° Fahr. (163° Cent.) (per cent.) . . .	Max. 2·0	Max. 2·0

NOTE.—The above tests shall be performed in conformity with the methods set out in the current edition of "Standard Methods of Testing Petroleum and its Products," published by the Institute of Petroleum.

The properties of alternative asphaltic cements as permitted in the Note to Clause 1 shall be determined in the following manner :—

Column (b), Table 1, is based upon equal proportions of asphaltic bitumen (ash free) and refined Lake asphalt (ash 36 per cent.). Varying proportions of these two components within the limits specified shall be permissible if the resulting asphaltic cement has characteristics coming within the provision of column (b) above, except that the values for soluble bitumen and mineral matter will be proportional to the percentage of refined Lake asphalt present.

Part 4. Mastic Asphalt for Roofing (Limestone Aggregate)

5. *Mastic Asphalt.*—The mastic asphalt shall be a mixture of asphaltic cement and limestone in proportions necessary to conform to the requirements of Clause 8 of this Specification.

6. *Limestone.*—The limestone shall be ground, and when tested in accordance with the methods specified in Clause 3, the grading shall conform to the values given in Table 2.

Table 2.—*Grading of Limestone*

	Percentage by weight.	
	Min.	Max.
Passing 200 mesh sieve	45	55
Passing 85 mesh sieve and retained on 200 mesh sieve	10	25
Passing 36 mesh sieve and retained on 85 mesh sieve	10	25
Passing 8 mesh sieve and retained on 36 mesh sieve	15	20

7. *Grit.*—The grit, if any, shall be incorporated during manufacture and shall not exceed 20 per cent. by weight of the total mastic asphalt.

It shall contain not more than 15 per cent. by weight of constituents soluble in dilute hydrochloric acid, and when tested in accordance with the method specified in Clause 3, shall pass a $\frac{1}{8}$-inch mesh sieve and at least 85 per cent. shall be retained on an 18-mesh sieve.

8. *Characteristics of the Mastic Asphalt.*—The properties of the mastic asphalt in the ungritted state shall show on analysis a composition within the limits given in Table 3.

The limits soluble of asphaltic bitumen and the grading of aggregate will be altered proportionately by the percentage of grit added.

TABLE 3.—*Composition by Analysis of Mastic Asphalt*

	Percentage by weight.	
	Min.	Max.
Soluble bitumen	11·5	14·5
Passing 200 mesh sieve	38·0	48·0
Passing 85 mesh sieve and retained on 200 mesh sieve	9	20
Passing 36 mesh sieve and retained on 85 mesh sieve	9	20
Passing 8 mesh sieve and retained on 36 mesh sieve	12	17

9. *Hardness Number.*—The hardness number determined at 77° Fahr. (25° Cent.) shall be not less than 30 for the mastic asphalt as laid.

PART 5. RECOMMENDATIONS FOR APPLICATION

Remelting on Site of Work.—The mastic asphalt blocks should be broken into pieces of convenient size, and then carefully remelted in cauldrons or mixers, or when more convenient for the execution of the work it may be delivered to the site in a melted condition in mechanically agitated mixers; it shall be laid without its characteristics, as defined in Clauses 8 and 9, being impaired. The temperature of the molten asphalt shall at no time exceed 420° Fahr. (215° Cent.).

When removing the melted asphalt from the cauldron or mixer, the buckets should either be dipped in water or dusted. Ashes should not be used for dusting.

Laying Roof Covering.—The mastic asphalt should be applied uniformly on the prepared foundation, to the specified thickness.

Falls.—Where it is desired that flat roofs be kept free from standing water, a minimum constant fall of 1½ inches in 10 feet (1 in 80) should be provided in the foundation by the General Contractor.

Rubbing.—On flat surfaces and slight slopes, the finished asphalt should be well rubbed with a rubbing sand of naturally occurring quartz particles which may be derived from bank, river, dune or pit. The maximum size of such particles should not exceed $\frac{1}{8}$ inch (3·2 mm.).

Isolating Membrane.—An isolating membrane of black sheathing felt or other similar material, should be used on all horizontal surfaces, whether concrete, timber, over a thermal insulating medium and particularly when the asphalt is laid in one coat.

The membrane should be laid loose and not sealed to the foundation and should be dry when the asphalt is applied.

Vertical and Sloping Surfaces.—Vertical concrete surfaces or slopes (other than skirtings, risers, etc.) should, as a general rule, be prepared to receive the asphalt by adequate grooving or hacking.

On vertical and sloping timber surfaces, and at angles formed by, or at junctions with, such surfaces, in addition to a membrane, if any, expanded-metal lathing or other support should be provided to give an adequate key for the asphalt.

Asphalt Finishes.—In general, the following conform with accepted practice :—

Skirtings, aprons and other similar items should be executed in two coats in all roofing work, irrespective of whether the horizontal asphalt is laid in one or two coats.

(*a*) *Flashings.*—In order to ensure the full specified thickness, and to avoid local weakness at the edges of eaves gutters, etc., flashings should be set in a rebate formed in the foundation extending back at least 3 inches from the rounded edge of the flashing.

(*b*) *Skirtings.*—Skirtings should be formed to a height of not less than 6 inches and turned into a suitable horizontal chase provided by the General Contractor, and afterwards pointed.

(*c*) *Aprons to Eaves.*—Aprons should be finished with a drip edge.

(*d*) *Fillets.*—Special care should be taken to ensure that the asphalt angle is thoroughly clean before the forming of fillets, which is an independent operation.

(*e*) *Other Finishes.*—Metal, glazed, or other unusual surfaces should be specially treated before the asphalt is applied.

Horizontal Surfaces

Two-coat Work.—Where so specified, the asphalt should be applied in two coats to a total thickness of not less than $\frac{3}{4}$ inch, breaking joint at least 6 inches.

One-coat Work for Emergency Roofing.—Where so specified the asphalt should be applied in a single coat to an average thickness of $\frac{1}{2}$ inch on an isolating membrane.

Asphalt bays should where practicable be not greater than 4 feet 6 inches in width. The joints between bays should in no case be coincident with the joints in the isolating membrane and should be arranged as far from them as possible.

Strips of asphalt should be laid over the isolating membrane to a width of 9 inches along the lines where the subsequent joints in the asphalt will occur.

Particular care is necessary in workmanship, to compensate for the absence of the normal safeguards inherent in two-coat work. Special attention should be given to the sealing of joints.

Attention is called to the fact that where there is a thick isolating membrane the joint should be butted.

Part 6. Treatment of Surfaces with Paint for Camouflage or Other Purposes

For camouflage paint suitable for use on mastic asphalt roofing, see B.S. 987.

Great damage has been done to asphalt roofs by the use of ordinary paints.

CHAPTER IV

STONES

Classification.—Stones are divisible by the geologist into three chief classes: (*a*) Igneous, (*b*) Aqueous, (*c*) Metamorphic. Stones may be further divided for building purposes by those engaged in the arts into (1) sandstones, or those in which silica constitutes the base; (2) limestones, in which carbonate of lime forms the base; (3) slates; and (4) granites.

(*a*) *Igneous* rocks are of volcanic origin, having been formerly in a state of fusion, and include the granites, traps, and syenites.

(*b*) *Aqueous* rocks are those which have been deposited or formed in water or air, and include most of the limestones and sandstones in use for building purposes.

(*c*) *Metamorphic* are rocks of either of the above divisions which in many cases have been subjected to great heat or pressure, or both, sufficient to cause alteration in form, and may be simply a rearrangement of the particles as in clay slate; or a crystallization of the constituents of the producing rocks which have been subjected to great heat and pressure, such as the marbles; or the addition of new substances from solution in water percolating through rocks and producing new crystalline minerals such as the dolomites.

Stones vary from the stratified to the granular in structure. The stratified are those sedimentary rocks formed by successive deposits of the materials of which they are composed.

The stratification in good specimens should not be visible to the naked eye, unless through the difference of colour; the grains should also be uniform in size. Distinctly stratified stones are useful for pavings, landings, etc., as they may readily be split along their planes of stratification, called in this case planes of cleavage.

The granular are those that have been formed by volcanic agency, or those sedimentary rocks whose original structure has been altered by oscillations of the earth's surface and the action of fire or hot water.

Characteristics of Building Stones.—The salient characteristics of stones will be treated under the following heads :—

General Structure, Fineness of Grain, Compactness, Porosity and Absorption, Weight, Appearance, Seasoning, Natural Bed, and Weathering.

General Structure.—Sandstones consist of grains of sand cemented together by one or more of the following : silicic acid (H_2SiO_3) calcium carbonate, magnesium carbonate, peroxide of iron and clay.

Limestones usually consist of crystallized grains of calcium carbonate joined together by a cement of the same material, and when these are capable of taking a polish are termed marbles ; or, as in the oolite, of a number of grains, which are formed of calcareous matter deposited about a nucleus, the latter being usually small shells.

Marbles proper consist of a crystalline granular aggregate of calcite, white when pure and having the texture of loaf sugar, but passing into various colours according to the nature of the impurities. It occurs in beds among the schists, and is no doubt a limestone, formed either by chemical precipitation or by organic agency, which has been thoroughly metamorphosed by heat and pressure into its thoroughly crystalline character. Some of the fossiliferous limestones through which the Christiania granite rises have been changed into marble are crystalline solids, but their original corals and shells have not been wholly effaced.

Dolomites are stones in which the chief constituents are calcium carbonates and magnesium carbonates, and are considered most durable if these compounds are in nearly equal quantities. These are compact, crystalline, and oolitic in structure, and are superior to ordinary limestone.

The consolidation of these materials into solid rock has been accomplished by some or all of the following causes :—

Partial solution or redeposition; and heat, either dry or moist. Pressure of overlying water or rock.

Stones are termed freestones if granular in structure, with no planes of cleavage, and therefore no tendency to split in any direction, and for that reason are useful for carved work.

Granites consist of a crystalline granular structure, and are very difficult to work.

Fineness of Grain.—Fine-grained freestones are in great demand for carved or moulded work, as it is possible from these to obtain much finer arrises than from the coarser-grained varieties. Such stones depend for their durability upon the extent of the crystallization of the particles, and the quality of the cementing material; but if the particles are amorphous and of an earthy appearance, they are bad, and will be readily disintegrated by any of the destructive agents.

Compactness.—The durability of stones to a large extent depends on the compactness of the particles or density of the stone. For this reason the best building stones are those of the older formations formed at a great depth, and having been subjected to the enormous pressure of the earth above, and are very compact. These stones are often found near the surface, due to alterations of the earth's crust from internal causes, such as volcanic eruptions, earthquakes, etc., or to denudations from external changes due to wind, running water, glaciers, etc., which wear away the upper crust of the earth, and expose the rocks of the older formations.

Porosity and Absorption.—All stones are porous, but some to such an extent as to render them unfit for building purposes, especially for structures in exposed situations, although the constituents and cementing material may be of a durable character.

Porous stones may be destroyed in one of two ways, or by both combined: (*a*) by decomposition, (*b*) by disintegration.

Porous stones absorb much rain-water, especially when the faces of the buildings are exposed to the prevailing winds.

(*a*) Rain-water in its descent takes up some of the acids present in the air; these acids, chiefly sulphuretted hydrogen (H_2S), hydrochloric acid (HCl), and sulphurous acid (H_2SO_3), especially the latter, exist in appreciable quantities in large and manufacturing towns. H_2SO_3 is not stable in the free state and soon changes to H_2SO_4, sulphuric acid, by absorption of oxygen from the air. The rain lodges on the surfaces of the stones into which it soaks, being often driven in by the wind; the acids combine with the constituents of the stones, dissolve them and cause the stones to crumble.

(*b*) In winter time, the water absorbed by the stone freezes, expands, disintegrating the particles and detaching portions of the surface. The adhesion of the particles in some stones is sufficient for a time to resist the expansive force of water when freezing; but even these give way in time to the action of successive frosts, especially when added to this force the stones are subjected to the effects of acids.

Stones should be tested for porosity by soaking samples in water, and noticing the amount they absorb.

Sandstones should not after 24 hours' immersion absorb more than 10 per cent. of their volume of water; limestones not more than 17 per cent.; granites not more than 1 per cent.

Weight of Stones.—The weight of stones should be taken into account, and they should be selected to suit the work to be executed. Heavy stones are required for buttresses, retaining walls, and marine structures, while for vaulting and similar work light stones are preferable. Weight is also an indication of the density, and therefore the porosity of a stone.

Appearance.—In the choice of a stone for a building the colour is a good guide as to durability. Highly-coloured stones are often preferred for their architectural effect, frequently at the expense of their durability. The red and brown shades of colour in all the sedimentary rocks are due to oxide of iron, which, if present in large quantities, is apt to disfigure the face of the stone by rust stains, and also leads to rapid disintegration; therefore, the lighter shades of any particular stone should be preferred to the darker

tints. There should be no clayholes, bands, or spots of colour whatever, but the stone should be uniform in colour and in structure.

Seasoning.—All stones when freshly quarried contain a quantity of moisture known as quarry sap, which renders the stone soft and makes it easier to cut; therefore all work should be placed upon the stone as soon as convenient after quarrying.

Stones gain considerably in hardness by being seasoned.

Stones when once worked to a finished surface should not afterwards have their dressed faces disturbed, as is often done on the cleaning down of a building; but should be, when bedded, covered with a wash of plaster of Paris and lime in about equal proportions, which can be easily washed off and with it all the dirt and stains incident to building work, as the quarry sap when drying out leaves a hard crystalline skin on the face which will weather considerably better than any fresh face formed after the removal of the original worked surface. This applies with especial force to limestones.

It is important that the sap should be expelled before the stone is placed in a building, because when fixed it cannot dry out so quickly, thereby making it subject to disintegration by frost, easier for acids to act upon it, and being in a soft condition it is liable to break should any great weight be placed upon it.

Stones should therefore be left, after quarrying, to season for a considerable time, which is best accomplished by leaving them in the open air, say for 6 to 12 months, in order that they may be freely acted upon by the sun and wind. They are often placed under cover in a shed with no walls to allow a free access of air and to protect them from rain.

Natural Bed.—The natural bed in a stone is that surface on which the material was originally deposited, but is not necessarily horizontal as it rests in the quarry, the strata being often inclined and even upright, due to the beds having become folded and disturbed by volcanic and other agencies.

The purpose for which a stone is used determines the position of the natural bed, as this has an important effect upon its durability. Stones must be arranged so as to obtain—first, the maximum strength to resist crushing; and secondly, to offer the greatest resistance to disintegration by frost. This is done in the following manner: (1) By placing the stone so that its laminæ are at right angles to the pressure, because the stone is much stronger for weight-carrying purposes when in that position than when the pressure is applied to the end grain of the stone; (2) if the laminæ of the stone be placed parallel to the face of the building, they will scale off successively from the effects of each succeeding frost, every lamina that peels off exposing a fresh face. The stones must therefore be placed with the plane of their laminæ at right angles to the face of the wall.

In walling, such as ashlar work, the laminæ are placed horizontally.

In strings and cornices, with undercut mouldings, the laminæ should be placed vertically, as if placed horizontally the laminæ would be likely to scale off. This principle cannot be carried out in the quoin stones of cornices and strings; if it were so bedded, the whole projection on the return face would after a few years have crumbled away. Such stones must be specially selected and be without any apparent stratification, and laid on their natural bed. Cornices bedded with their laminæ vertically should be covered with lead, as water is readily conducted between the layers.

The laminæ in arches should be placed parallel to the centre line of the voussoirs, and at right angles to the face of the arch.

In good qualities of stratified stone the beds are not easily discernible, and require a practised eye to determine their direction. They may be easily detected in some stones by thin bands of a greenish or blackish colour of vegetable origin. The beds may often be determined by pouring a little clean water on the stone and noticing the direction it takes in descending. If the stone be examined through a powerful magnifying glass, the particles of which the stone is formed will sometimes be observed to be flattened

in one general direction ; these flattened surfaces are usually parallel to the bed of the stone ; the planes of the minute flakes of mica occurring in most sandstones will be observed to lie in one general direction which indicates the natural bed. Experienced masons may discover by the feel when working the stone by the chisel the lie of the natural bed.

Weathering.—The weathering of a stone is the extent to which its face will resist the action of the weather.

It will be noticed that those faces exposed to the prevailing wet winds (south-west in England), and those that get most saturated with rain, are the faces that show the signs of decay to the greatest extent, also shady parts, such as the underside of cornices, etc., that are at no time exposed to the sunlight, and which never get the moisture dried out of them, being consequently left to the rain with all its attendant defects.

The best way to determine the weathering qualities of a stone is to inspect buildings in the neighbourhood of the site of the proposed building that have been built with the stone in question, or visit the quarry, and especially see any faces of the quarry that have not been used for a great length of time, and observe how they have weathered. If the stone be required for a building in a large town, the latter two observations will not be sufficient data to judge of its weathering qualities, but a chemical test as hereafter described will be necessary.

Constituents of Stones.—In order to understand the chemical action of the atmosphere upon stones, a knowledge of the constituents of stones is indispensable.

The composition of the principal classes of stones used for building work is given in the following order : Granite, sandstones, limestones, slates.

Granite.—Typical granite is composed of quartz, felspar, and mica ; the latter is frequently present in but very small quantities, with occasionally another mineral (hornblende).

Syenite.—Typical syenite is composed of felspar and hornblende.

Hornblendic Granite.—Those rocks containing the four

minerals—quartz, mica, felspar, and hornblende—are termed hornblendic granite.

Basalt is a compact black rock, consisting chiefly of felspar, augite, olivine, magnetite, and titaniferous iron embedded in glass or crystallites.

Diorite, sometimes termed "greenstone," is a basic rock containing plagioclase felspar and hornblende or some other ferro-magnesian silicate often associated with free quartz. These rocks are extensively quarried in Guernsey for road material, for which purpose diorites are most satisfactory.

Sandstones are composed of grains of silica cemented together by silicic acid or by calcium carbonate. Nearly all sandstones contain oxide of iron, to the presence of which they owe their colour. Besides these, sandstones often contain mica and clayey matter.

The stone may consist of grains of sand cemented together by lime or other material; the durability of the stone here depends on the quality of the cement, as the sand is indestructible; or, secondly, it may consist of particles of calcium carbonate or other substances joined together by a siliceous cement, in which case the grains are likely to decay, leaving only the cement, resulting in a porous stone. The most durable sandstones are those formed of grains of silica, cemented together by silicic acid, with but a small quantity of other matters, such as the Craigleith, which contains about 98 per cent. of silica and only 2 per cent. of impurities.

Limestones consist chiefly of calcium carbonate, with small portions of silica, magnesium carbonate, iron and clay, and are stratified or oolitic in structure.

Stones consisting of calcium carbonate and magnesium carbonate in nearly equal quantities are known as dolomites or magnesium limestones. Where these two compounds exist in stones in a crystalline condition, the stone is very durable.

Limestone also exists as gypsum or calcium sulphate, in

a crystalline condition as alabaster, and is burned largely for plaster.

Slates are obtained from the Devonian, Silurian and Cambrian strata. It is a compact, fine-grained argillaceous rock that has been subjected to enormous pressure, and also to a shearing action which has caused planes of cleavage, independent of the original beds, often crossing them at a great angle. It is on these planes of slaty cleavage, as they are called, that the value of slates depends, as this enables them to be split with facility into thin laminæ, and thus form a light covering. They are composed chiefly of silica and alumina.

They vary in colour from purple to green, and are used in all parts of the kingdom for roofing purposes.

The following gives the analyses of some of the well-known building slates, from Howe's " Geology of Building Stones " :—

	Delabole, Cornwall. Grey.	Maen-offeran, Wales. Dark Blue, Grey.	Moel-Tryfaen, Wales. Reddish.	Penrhyn, Wales. Purple.	Velenhelli, Wales. Blue.	Oakley, Wales. New Vein, Dark Blue.
SiO_2 ...	56·75	53·92	60·17	63·01	63·30	53·40
Al_2O_3 ...	21·81	24·09	18·89	16·28	14 99	26·67
Fe_2O_3 ...	1·36	1·87	6·17	8·42	8 75	9·53
FeO ...	7·20	6·52	0·95	0·66	1·53	—
MnO ...	trace	0·19	0·20	0·12	0·25	—
TiO_2 ...	0·71	0·80	1·15	0·58	1·04	—
CaO ...	0·84	0·24	1·75	0·72	1·08	0·90
MgO ...	2·48	1·80	1·85	1·95	1·63	1·85
K_2O ...	3·72	3·64	2·76	2·98	2·84	—
Na_2O ...	0·97	0·74	1·39	2·14	1·92	—
CO_2 ...	—	—	1·04	—	—	—
P_2O_5 ...	trace	0·19	0·11	trace	0·07	—
Carbon...	—	—	—	—	—	—
H_2O ...	4·23	5·87	3·70	2·95	2·88	—
BaO ...	—	—	0·04	—	—	—
SO_3 ...	—	—	—	—	—	—
FeS_2 ...	—	0·13	—	—	—	—
Loss on Ignition						4·47
Alkalies						3·18
	100·07	100·00	100·17	99·81	100·28	100·00

Characteristics.—A good roofing slate should be uniform in colour and free from patches, compact and sonorous, incapable of absorbing or retaining much water, hard and rough to the touch ; those which feel smooth and greasy, or are purple in colour, being usually inferior for roofing purposes.

Tests.—A common test for roofing slates is to place one on edge to half its depth in water for 12 hours. If the water approaches the top of the slate it should be rejected ; if it does not rise beyond $\frac{1}{3}$ inch, it may be considered as practically non-absorbent. Another method is to weigh a well-dried slate, and after soaking for 12 hours in water to weigh it again ; the difference in weight will show the quantity absorbed.

A good slate after 12 hours' soaking should not have absorbed more than $\frac{1}{200}$th part of its weight.

Chemical Composition.—The following is the chemical composition of the principal constituents of stones :—

Quartz is silicon oxide (SiO_2) ; it is practically indestructible, and is often found coloured owing to the presence of small quantities of impurities, generally metallic oxides.

Felspar is a silicate of aluminium, with silicates of sodium or potassium, or a mixture of the two.

$$K_2O \cdot Al_2O_3 \cdot 6\ SiO_2 \text{ (Orthoclase)}$$
$$Na_2O \cdot Al_2O_3 \cdot 6\ SiO_2 \text{ (Albite)}$$

the K or Na may be replaced by equivalent quantities of Ca, Mg, or Fe(ous).

$$CaO \cdot Al_2O_3 \cdot 2\ SiO_2 \text{ (Anorthite).}$$

Plagio-clastic felspars are usually mixtures or solid solutions of Albite and Anorthite in varying proportions.

Felspar often contains small quantities of oxide of iron, on which ingredient the colour of red granites depends. If it contains a large proportion of calcium, sodium or potassium, it will be liable to decay.

Mica is found in thin, hard, transparent plates or laminæ, which readily split ; it consists chiefly of silicate of

aluminium with potassium. Chemically there are two classes: first, the alkali micas, often called white mica, represented by the formula for muscovite—potash mica—$H_3K \cdot AL_3 (SiO_4)_3$; second, ferro-magnesium mica, often called black mica, represented by the formula for biotite—$K_2HMg_6Al_3 (SiO_4)_6$. It imparts the glistening appearance to granite, is readily decomposed, and in large quantities is a source of weakness.

Hornblende is a silicate of calcium and magnesium. These two constituents being in varying proportions, 5 (Mg, Ca)O . 6 SiO_2; it is very heavy and of a black or green colour.

Silicic Acid.—$(H_2O)_x(SiO_2)_y$ forms the best cementing material for all sandstones.

Augite is a similar substance to hornblende, (Mg,Ca) SiO_3.

Calcium Carbonate.—The basis of all limestones, $CaCO_3$.

Magnesium Carbonate.—$MgCO_3$ exists in numbers of limestones. When the stone is approximately of the following composition it is called a dolomite: 54 $CaCO_3$ + 46 $MgCO_3$, or (Ca,Mg)CO_3.

Gypsum is a hydrated calcium sulphate, $CaSO_4 \cdot 2\ H_2O$.

Alumina.—Al_2O_3 is an oxide of the metal aluminium; combined with silica it forms the basis of clay.

Kaolin.—$Al_2O_3 \cdot 2\ SiO_2 \cdot 2\ H_2O$ is a pure white clay derived from the decomposition of felspar, which has been acted upon by water containing carbonic acid, which latter dissolves the calcium, potassium, and sodium, leaving the silica, alumina, and water.

Chemical Tests.—Immerse a few chippings of the stone in a 5 per cent. solution of dilute sulphuric or hydrochloric acids for 3 days. When taken out and dried the surface grains should still be firm and the angles sharp. Loose sand

about the surface would indicate a speedy dissolution in a town atmosphere.

A few drops of the pure acids dropped on to a sandstone would, if effervescence took place, indicate calcium carbonate as a constituent, probably as the cementing material. Such a stone would not weather well.

The following is a list of the stones most largely used :—

LIMESTONES

Ancaster.—An oolite quarried in Lincolnshire. Composed of :—

Calcium Carbonate	93·59
Magnesium Carbonate	2·9
Iron and Alumina	0·8
Water and loss	2·71

The colour varies from cream to brown. The weight per cubic foot for the cream coloured, a fine oolite is 136·3 lbs. For the brown a compact shelly stone which weathers well is 156·3 lbs. Most of the Lincolnshire churches are built of this stone ; it is used for local building work generally, in London and the Midland counties for dressings to doorways, windows, etc.

Box Ground, Corsham, and Coombe Down are varieties of the Bath oolite. Composed of :—

Calcium Carbonate	94·52
Magnesium Carbonate	2·5
Iron and Alumina	1·2
Water and loss	1·78

Varies from a light cream to a yellow colour. Of the three the Box Ground is the best weathering stone. Its weight per cubic foot is 127·9 lbs. It is largely used for dressings, carved and moulded work.

Bath stone to endure must be kept dry and be laid on its natural bed.

Fine-grained Corsham with a weight of 129 lbs. per cubic foot is good for interior work and external work well above the ground level. In the 6 months of winter the stone is dug and stacked under ground, and is then brought to the surface and seasoned for a period such as 6 months, before being used.

Box Ground is good in all situations, but if used for walls must be at least 18 inches thick to prevent damp passing through. It is quarried in the severest weather, and withstands all injury from the weather. Exposure causes the quarry sap to evaporate, and the stone hardens, and therefore gets more difficult to work.

Monks Park.—Quarried at Corsham, is an oolite of a whitish cream colour, compact, close grained, it weighs 136·7 lbs. per cubic foot and its ultimate resistance to crushing is 139·6 tons per square foot, and is among the best of the Bath stones.

Portland.—An oolite from the Isle of Portland. Composition as follows:—

Silica	1·2
Calcium Carbonate	95·16
Magnesium Carbonate	1·2
Iron and Alumina	0·5
Water and loss	1·94

Its weight per cubic foot is 134·1 lbs. The colour varies from a white to a light brown, the latter being considered the better. There are four beds of Portland, the true Roach, Whitbed, bastard Roach, and Basebed, the Roach and Whitbed being the best. Portland stone is used for building work generally, and is found to weather better than all other limestones for large towns.

There are three districts in Portland—Wakeham, Minesthay, and Weston—which supply stone in marketable quantities. Their properties seem generally to vary considerably. The first is good to withstand atmospheres charged with sulphuric acid, such as Birmingham; the second lasts well in sea-coast districts, such as Eastbourne and Portsmouth, and stands well in London. Stone from the third district is good in moist and forest atmospheres, and especially north and west of England and Ireland; for important buildings it would undoubtedly be wise to consult the quarry owner in the selection. Although it is safe to lay Portland stone on its natural bed, it is not so important as in Bath and other limestones.

Ham Hill is obtained from Somersetshire. Composition as follows :—

Silica	4·7
Calcium Carbonate	79·3
Magnesium Carbonate	5·2
Iron and Alumina	8·3
Water and loss	2·5

Its weight per cubic foot varies from 136 to 141·5 lbs. Colour, yellow and grey; the first is bright when first quarried, but tones down. Used for facings and dressings, weathers well.

Chilmark.—A brown oolite, obtained from Wiltshire. Composition as follows :—

Silica	10·4
Calcium Carbonate	79·0
Magnesium Carbonate	3·7
Iron and Alumina	2·0
Water and loss	4·2

Its weight per cubic foot is 155·1 lbs. Light brown in colour. Very durable. Used for Salisbury Cathedral, suitable for general building, specially steps and paving, and much used for heavy engineering work.

Doulting Freestone.—An oolite of uniform texture, obtained near Shepton Mallet, Somersetshire. Composition as follows :—

Calcium Carbonate	95·89
Magnesium Carbonate	0·11
Alumina	0·79
Ferric Oxide	0·85
Silica	2·04
Water	0·32

Its weight per cubic foot is 153·1 lbs. It varies from a cream to a brownish-yellow colour. It is very durable, and suitable for general building work.

Little Casterton.—An open oolite quarried near Stamford; used locally for general building purposes.

Caen.—A fine oolite quarried at Caen, in Normandy. It has been largely imported into England, but is a failure for

external work, as it weathers very badly, but is much used for internal work for decorative purposes, being well adapted for carving.

Nailsworth.—A brown oolite quarried at Nailsworth, Gloucestershire, similar to Portland stone, but considerably cheaper. The beds of rock are entirely free from any defects and can be obtained in huge sizes. Very mild working when freshly quarried, but hardens quickly.

It is recommended for internal work.

Irish Limestones

Skerries.—Quarried at Skerries, County Dublin. Much used in the neighbourhood ; a good building stone. Colour grey to black.

Weight per cubic foot, 172 lbs.

Tullamore.—Quarried at Tullamore, King's County, Ireland. In colour a light blue appearance when finished by the chisel, black when polished. Weight per foot is 168 lbs. Sometimes this stone is classed as a marble.

Cairns Lodge.—Quarried at Tyrrell's Pass, County West Meath, Ireland. One of the best of Irish limestones. A large tract still unworked. In colour, grey. A good durable building stone. Weight per foot 166·5 lbs.

Crossdrum.—Quarries at Oldcastle, County Meath. Good working and durable building stone. Colour, light grey. Weight per cubic foot 166·0 lbs.

Little Island.—Quarried at Little Island, Co. Cork. In colour, light grey. Weight per cubic foot 165 lbs. A good working stone. Used for forts, Spike Island.

Magnesium Limestones or Dolomites

Anston.—Quarries near Sheffield, Yorkshire. A magnesium limestone from the Permian bed, is a fine granular crystalline aggregate with cavities. Dispersed through the mass are black particles, apparently carbon. This is probably not an altered limestone, as the structure of the stone indicates precipitation from chemical solution. It is

of a rich cream colour. Weight when dry 141 lbs. 3½ ozs., when wet 148 lbs. 14½ ozs. per cubic foot.

Calcium Carbonate	54·87
Magnesium Carbonate	43·07
Alumina and Iron Oxide	0·73
Silica	0·56
Water and loss	0·75
	99·98

It has been largely used in the Houses of Parliament, Westminster, Geological Museum, Piccadilly, and the Record Office, Chancery Lane. The beds are from 1 foot to 2 ft. 6 in. thick, and they are most durable when laid on their natural bed. Test pieces of this stone cracked slightly with 815·6 tons, and crushed at 833·1 tons per square foot.

Bolsover.—From Derbyshire. Composition as follows :—

Silica	3·6
Calcium Carbonate	51·1
Magnesium Carbonate	40·2
Iron and Alumina	1·8
Water and loss	3·3

Its weight per cubic foot is 141·7 lbs., and it possesses a pleasing light yellowish-brown colour. Good durable building stone, also useful for paving.

Mansfield Woodhouse.—Obtained from Nottinghamshire. Composition as follows :—

Silica	3·70
Calcium Carbonate	51·65
Magnesium Carbonate	42·60
Water and loss	2·05

Its weight per cubic foot is 145·6 lbs. Yellow colour. Much used for internal decorative work, does not stand well externally.

Huddlestone.—Quarried at Sherburn in Yorkshire. Composed as follows :—

Silica	2·53
Calcium Carbonate	54·19
Magnesium Carbonate	41·37
Iron and Alumina	1·3
Water and loss	1·61

Its weight per cubic foot is 133·7 lbs. Cream colour. Suitable for general building purposes.

2,200 tons per square foot (Kirkcaldy). Absorption, 1·02 per cent. Composed as follows :—

Silica	97·83
Alumina or Oxide of Iron	1·17
Lime	1·00
	100·00

Hailes.—The stones from these quarries near Edinburgh, in the neighbourhood of Craigleith, are of three tints, white, pink, and bluish grey.

The white rock appears, from its more complete secondary silicification, to be the strongest and most compact in structure of the three varieties. The small percentage of iron and of the ferrous compounds points to a probability of greater immunity from discoloration and liability to rust stains than the blue rock. It is finer in grain and on all accounts to be preferred to the blue rock. The following gives the analysis of the white rock :—

Silica	96·52
Lime	0·409
Sodium Oxide / Potassium Oxide	0·27
Ferrous Oxide	0·19
Ferric Oxide	0·056
Alumina	2·78
Loss on ignition	0·31
	100·535

The blue rock has the most marked lamination, and the cementitious matter is partly aluminous. The blue colour is due to fine lines of carbonaceous matter. The following is the analysis :—

Silica	92·23
Lime	0·81
Magnesia	0·19
Sodium Oxide / Potassium Oxide	0·18
Ferrous Oxide	1·71
Ferric Oxide	0·013
Alumina	2·93
Loss on ignition	1·73
	99·793

The pink is a variety having properties intermediate between the white and the blue. The following is the analysis :—

Silica	96·7
Lime	0·36
Magnesia	trace
Sodium Oxide / Potassium Oxide	0·13
Ferrous Oxide	0·25
Ferric Oxide	1·46
Alumina	0·84
Loss on ignition	0·58
	100·32

Stone from this quarry has been used largely in Edinburgh and neighbourhood for the last two centuries, and has proved an excellent weathering stone for general building purposes. It is especially suitable for templates, steps, landings and pavements, and can be obtained in very large blocks. The following test for transverse strength was made on this stone at the New Art Galleries, Kelvingrove, Glasgow. The steps tested were each 11 ft. 6 in. long, having a 9-inch wall hold at each end. Three steps were built into walls having a clear span of 10 feet; they were then gradually loaded on top with steel joists till the load reached 6 tons. The deflection at centre of steps was $\frac{3}{8}$ inch, and the loading occupied a space of 4 feet in the centre of the steps. The intention was to load until they broke, but when the results were so satisfactory the steps were saved.

Bramley Fall.—Originally quarried near Leeds, but the name is now used to denote the coarse millstone grits of Yorkshire from quarries such as Horsforth. Its weight per cubic foot is 131·9 lbs. Light brown colour. Very durable and good for general building purposes. Specially suitable and much used for pavings, steps, girder beds, engine beds, etc.

Darley Dale.—Quarried near Bakewell, Derbyshire. Composed as follows :—

Silica	96·4
Calcium Carbonate	0·36
Iron and Alumina	1·3
Water and loss	1·94

Weight per cubic foot is 163·7 lbs. Light brown colour. Good for general building purposes, and is largely used.

Professor J. Shipman, F.G.S., in his report upon the best building stones, says: "The excellent quality of the stone from the Stancliffe Quarries, Darley Dale, is seen in the entrance lodge and gates of the park (to Stancliffe Hall), which are built of it. The work presents a superbly clean and crisp appearance, the most delicate sculpturing and cornicing standing out as sharp and clear as if it had been chiselled the previous day. The stone is of a light drab or yellowish-white colour, inclining to a very pale greenish tint, but the colour of the stone is so subdued that it is almost white. There are no streaks or blotches of red about it, and its texture is very uniform throughout. When very closely examined it seemed to be a close-grained, finely micaceous grit, the mica occurring in only very minute silvery spangles.

"The rock is a thick-bedded grit, compact and very hard. It has been largely quarried, and is considered by the authorities to be a most valuable stone.

"St. George's Hall, Liverpool, is perhaps the finest example of a building in which this stone has been employed."

Forest of Dean.—Quarried in Gloucestershire. There are two kinds, the grey and the red.

The grey is composed as follows:—

Silica	80·16
Alumina	14·40
Oxide of Iron	1·65
Calcium Carbonate	2·55
Magnesia	0·24
Calcium sulphate	1·00
	100·00

Its weight per cubic foot is 150·6 lbs., and it varies in colour from grey to shades of blue, hard and suitable for general building work, templates, etc.

The red is composed as follows :—

Silica	88·70
Alumina	3·25
Ferric Oxide	1·80
Ferrous Oxide	0·30
Manganese Oxide	1·10
Calcium Oxide	2·90
Magnesia	0·11
Carbonic Acid	1·94
Alkalies	0·31
Loss	0·59
	100·00

Varies from a light to a deep rich red, the lighter shades being extremely hard.

Heddon.—Quarried near Newcastle, Northumberland. Composed as follows :—

Silica	95·1
Calcium Carbonate	0·8
Iron and Alumina	2·3
Water and loss	1·8

Its weight per cubic foot is 143·1 lbs. Light brown colour. Durable stone, good for general building purposes.

Kenton.—Quarried near Newcastle, Northumberland. Composed as follows :—

Silica	93·1
Calcium Carbonate	2·0
Iron and Alumina	4·4
Water and loss	0·5

Its weight per cubic foot is 140 lbs. Light brown colour. Good for general building purposes, particularly for fine and carved work.

Grinshill Freestone.—Quarried near Yorton, Shropshire. Composed as follows :—

Silica	95·46
Alumina	1·17
Peroxide of Iron	0·87
Calcium Carbonate	0·61
Magnesium Carbonate	0·69
Water	1·2

Its weight per cubic foot is 122·5 lbs. Yellowish-brown

colour. A fine-grained soft sandstone, extensively used for facing and general building work.

Scotgate Ash.—Quarried at Pateley Bridge, in Yorkshire. Its weight per cubic foot is 153·1 lbs. Greyish-yellow in colour. It is fine-grained, hard, laminated sandstone, suitable for staircases, pavements, etc.

Howley Park.—Quarried at Morley, Yorkshire. Its weight per cubic foot is 160 lbs. Light brown colour. A fine-grained homogeneous sandstone, durable, not hard, used for dressings, stairs, pavings, and general building work.

Robin Hood.—Quarried near Wakefield, Yorkshire. Greenish-grey colour. Durable, suitable for landings, staircases, etc.

Corsehill.—From Annan, in Dumfriesshire. Its weight per cubic foot is 153·7 lbs. Dark red and bright pink in colour, contains about 95 per cent. of silica. Good weathering stone, suitable for carvings, dressings, and ashlar.

The chemical analysis is as follows :—

Silica	95·24
Alumina	0·56
Iron Oxides	1·28
Calcium Carbonate	1·40
Magnesium Carbonate	1·23
Water and loss	0·56
	100·27

Prudham.—Quarried at Fourstones, Northumberland, the thickness of the bed is from 40 to 50 feet and lies about 65 feet below the carboniferous limestone series. Its composition is as follows :—

Silica	86·4
Alumina	6·45
Iron Oxide	0·75
Lime	0·6
Magnesia	0·27
Alkaline Metals	0·21
Water	5·25
	99·93

It weighs in fairly dry condition 180 lbs. per cubic foot, is of a lightish cream colour, and is an excellent stone for dressings, moulded and ashlar work, and is largely used in the north of England and the south of Scotland. The value of the ultimate resistance to crushing is 455·3 tons per square foot.

Pennant.—This stone from the Pennant series in the coal measures of Glamorganshire and Gloucestershire, is of a dark grey, greenish or blue colour, is hard and strong, may be worked to a fine surface, weathers and wears well, is a good but expensive building stone, and is obtainable in large blocks, and is suitable for bed stones.

The chemical analysis of the stone from the Craig-yr-Hesg quarry is as follows :—

Silica	83·15
Alumina	8·10
Iron Oxides	4·54
Lime	0·38
Magnesia	0·68
Alkalies	0·78
Water and loss	2·37
	100·00

IRISH SANDSTONES

Mountcharles Stone.—Quarries at Mountcharles, Co. Donegal, Ireland. This stone is of a felspathic grit rather than a normal sandstone and was evidently from a granite rock. In colour it is a warm cream and bleaches white after a few years' exposure. It is one of the finest sandstones obtainable, and for appearance as well as weathering qualities cannot be surpassed for city conditions, and can be worked to any desired finish. Many of the largest and finest buildings in Dublin, Belfast, Londonderry and other parts of Ireland have been built of this stone. Labour about the same as that of Portland stone. All dimensions obtainable.

Weight 150·7 to 157·8 lbs. per cubic foot.

Chemical analysis :—

Silica	76·76
Alumina	11·13
Ferrous Oxide	0·15
Ferric Oxide	0·50
Lime	1·25
Magnesia	0·59
Potassium Oxide	3·72
Sodium Oxide	2·47
Loss of ignition	2·95
	99·52

Crushing strain (Kirkcaldy) :—

763·2 to 772·5 tons per square foot.

Shamrock Stone.—This stone is obtained from the Doonagore Liscannor Quarries, Co. Clare, Ireland. It is a hard, clean, close-grained, blue-grey, millstone grit; absolutely uniform in substance and colour, entirely free from nodules or other hard bits of grit such as are found in so many otherwise good stones. Admirably adapted for landings, steps, kerb-channelling, flagging of footpaths, etc.

These natural faced flags are exceedingly durable and will not wear slippery. Too hard to be dressed by machinery.

Weight per foot 168·2 lbs.

Used for circular staircase, St. John's Tower of London, Acid Testing Room, Royal Mint, and several other public buildings in London. Analysis :—

Silica	84·90
Alumina	6·60
Oxide of Iron	3·60
Oxide of Manganese	0·65
Lime	0·90
Magnesia	1·26
Sulphuric Acid	trace
Carbonic Acid	nil
Alkalies	0·39
Water	1·70
	100·00

Ballyvoy.—Quarried at Ballycastle, Co. Antrim. A fine grained sandstone, easily worked, finishes well. Used for general work. Light grey colour. Its weight per cubic foot is 140 lbs.

New Milling.—Quarries at Dungannon, Co. Tyrone. Light cream in colour. Its weight 137 lbs. per cubic foot. Works well. Used for dressings and general building work.

MAGNESIUM SANDSTONES

Red Mansfield.—Quarried near Mansfield, Nottingham. Composed as follows :—

Silica	49·4
Calcium Carbonate	26·5
Magnesium Carbonate	16·1
Iron and Alumina	3·2
Water and loss	4·8

Its weight per cubic foot is 145·6 lbs. Reddish-brown colour. Oolitic in structure, very durable. Suitable for general building work, carving, moulding, etc.

White Mansfield.—Quarried in the same district as Red Mansfield. Composed as follows :—

Silica	50·0
Calcium Carbonate	41·3
Magnesium Carbonate	7·3
Water and loss	1·4

Its weight per cubic foot is 145·6 lbs. Whitish-brown colour. Similar in all respects to the red variety, but is not considered so durable.

GRANITES

Igneous Rocks.—The igneous rocks as used for building purposes consist of granites and basalts which are of a crystalline granular structure.

Quarrying and Working.—In Great Britain granite is chiefly obtained from Cornwall and Aberdeenshire. As the British quarries are insufficient to supply the demand a great quantity is imported from Guernsey, Norway, Sweden and Russia, but the working of the granite from all parts is chiefly carried on at Aberdeen.

The method of quarrying is as follows: Blocks are detached by blasting, which are hauled from their position in the quarry by means of steel rope railways to the point at which they are worked. Blocks are roughly squared

into suitable dimensions by nicking a line about the stone and sinking small holes about 1 foot apart into which are inserted steel wedges and feathers; these are tapped with a hammer in succession until, the stress becoming too great, the stone is rent through the plane bounded by the nicking. The stones are also roughly faced at the quarries with hammers, and fine faced with hammers and chisels. Large surfaces are now more expeditiously faced with a surfacing machine. The block is placed in position and a large surfacing tool worked by pneumatic power is caused to strike the stone with a series of rapidly delivered blows, the tool being guided by the hand; the surface is gradually worn to a uniform level. Pneumatic chisels are used also for lettering, carving and moulded work. Granite is sawn into slabs by means of toothless steel blades assisted by chilled cast-iron shot, the stones being divided at the rate of about 2 inches per hour. The stones for polishing are placed on a table and bedded in plaster of Paris; the surface is rubbed and polished by means of heavy circular iron rubbers rotating on vertical axes, the table having imparted to it a reciprocating horizontal motion, thus permitting of the entire surface being rubbed uniformly, chilled shot, carborundum, sludge and putty powder being used as mediums to produce the polished surfaces. For moulded work, cast-iron slippers the reverse of the mouldings are caused to work over the members to be polished, which are embedded in plaster to preserve the arrises and mitres. Circular work is turned and polished in lathes.

Classification.—Igneous Crystalline Rocks are usually classified as follows:—

I. Acid rocks, with 65 to 80 per cent. of silica.

II. Intermediate rocks, with 55 to 70 per cent. of silica.

III. Basic rocks, with 45 to 60 per cent. of silica.

IV. Ultra-basic rocks, with 35 to 50 per cent. of silica.

Granites are included in Group I. Syenite proper, containing felspar and hornblende but no free silica, belongs to Group II. Basalts belong to Group III., whilst the rocks of Group IV. are too rare to be used for building purposes.

Stones comprised in the groups other than I. are often incorrectly termed granites.

Grey Aberdeen.—Quarried at several places in Aberdeenshire. Takes a high polish ; suitable for columns and ornamental work ; largely used for kerbs and sets.

Rubislaw.—One of the chief quarries is Rubislaw, near Aberdeen. It is fine grained, grey in colour, and can be obtained in blocks as large as 240 cubic feet. It takes a high polish, is extremely durable and is largely used for constructional, monumental and decorative purposes, and can be obtained in large quantities. The following is its composition, given by Professor Geikie :—

" This rock is a true granite. It contains white and black micas (muscovite and biotite), of which the latter is the more abundant, together with felspars and quartz.

" Of the felspars, microcline is the most prevalent, but plagioclase occurs in fair quantity. Fine needles of rutile are seen in the quartz. Other accessory ingredients sparingly present are zicron, apatite, and magnetite.

" This is one of the most durable kinds of granite."

Pink Aberdeen.—Various shades of this kind are quarried in Aberdeenshire. It answers the general description of the grey varieties.

A noted quarry is that of Correnie, the stone from which is of a close grained pink variety and is largely used for constructional and decorative work.

Peterhead.—The stone from this quarry in Aberdeenshire is of a coarse red variety and is largely used for constructional and decorative purposes. Many large polished columns have been executed in this material.

Cornish Granite.—Obtained in various parts of Cornwall. Grey in colour. Is largely used for engineering works, bridges, and similar constructions.

Guernsey Granite.—Quarried in Guernsey. Varies from reddish-brown to a grey-blue colour. It is a syenitic granite containing felspar, quartz and hornblende, and a little mica, used chiefly for paving sets.

Porphyritic Granite, Shap Fell.—The stone from this Westmorland quarry is of a reddish-brown tint and contains large pinkish crystals of felspar, takes a high polish. and is largely used for decorative work.

Irish Granites

Castlewellan.—Quarried at Castlewellan, Co. Down, North Ireland. Weight per cubic foot, 172 lbs. In colour a mottled grey. A fine and beautiful stone, one of the best granites in the isles. All sizes obtainable. Used for Eddystone and Blackbeat Lighthouses and base and pedestal of Albert Memorial.

Dalkey.—Arklow, Co. Wicklow. Weight per cubic foot 169·6 lbs. Bluish-grey colour. A good building stone.

Newry.—Newry, Co. Down. Weight per foot, 170 lbs. Grey colour. Suitable for general building purposes, is very durable and much used in the north of Ireland.

Dun Laoghaire.—Dun Laoghaire (Kingstown), Co. Dublin. Weight per cubic foot, 171 lbs. In colour, grey. Hard stone to work, and largely used for paving sets.

Foreign Granites

Swedish Granite, Victoria Grey.—This rock is quite a normal granite, and belongs to the acid group and contains orthoclase, biotite, and quartz. Along with these occur a little microcline and micropegmatite, apatite, and zicron. The last-named is enclosed in the biotite, and is surrounded by black halos. Is used for decorative and monumental work.

Finland Granite.—This rock is a fine hornblende—granitite. It has a schistose aspect—a structure which is not seen under the microscope.

The chief mineral constituents are—hornblende, biotite, orthoclase, plagioclase, and quartz.

The accessory and minor ingredients are apatite, magnetite, zicron, and sphene—the last-named being fairly

common. The hornblende is green, and the biotite is dark brown. Both occur in small crystals, usually associated and often well formed. The orthoclase is very fresh and abundant. Plagioclase is not very common. Quartz occurs in fair quantity. Epidote (a secondary mineral, or product of decomposition) is present.

It is a rock of the acid group, but more basic than the Victoria grey. Is used for decorative and monumental work.

Norway Granite.—A fine-grained granite quarried in Norway, extensively used for curbstones and pitchings; it is preferred on account of the great lengths in which it can be obtained.

Swedish Labradorite.—This is a rock of the intermediate group, and is not at rue granite. It is a greyish-green and very coarsely crystalline rock. To the naked eye it seems to consist chiefly of large felspars, showing some play of colours with a subordinate proportion of dark mica and pyroxene.

Under the microscope the felspars prove to be the varieties known to mineralogists as anorthoclase or cryptoperthite (that is, mixtures of albite and orthoclase).

The other ingredients are dark greyish-green augite with diallage-structure, deep brown biotite (mica), magnetite, and apatite, which appears in relatively large crystals. This rock belongs to the class of augite-syenites (known to geologists as laurvikites), which are well developed in Southern Norway. (The rocks referred to are somewhat variable in composition, containing often zicron; occasionally olivine and nepheline, and less commonly quartz.)

Used for monumental and decorative work.

Swedish Bon-Accord.—This is a rock of the third or basic group, and is not a true granite.

This rock is an olivine-gabbro. Its constituent minerals are plagioclase, felspar, augite (diallage), and olivine, with a small proportion of black mica and magnetite. This plagioclase occurs in fairly well-formed crystals which are occasionally enclosed in the augite—thus showing a ten-

dency to what is known as the ophitic structure so commonly seen in the rock called diabase. The olivine is very fresh and rather abundant. The black mica (biotite) mostly occurs in the form of scales around the magnetite.

Gabbros of this character occur at Elfdalen, in Sweden. Used for monumental and decorative work.

Granite does not successfully resist the action of fire or acids.

Preservation of Stone.—Of late years, a solution known as Fluate, and prepared by the Bath Stone Firms, has been introduced and extensively used to harden and preserve limestones in new work from decay, and also it is useful in preventing the decay in old work going farther. It does not materially alter the colour nor apparent texture of the stone to which it is applied, but it hardens the face and renders the stone more durable. The work is first cleaned and then the fluate is laid on the face with a brush.

Portland stone fronts exposed to severe weather are sometimes coated with boiled linseed oil, which effectively preserves the stone. After a lapse of 2 or 3 years the colour of the stone is darkened. There are several examples of this treatment in the Isle of Portland.

Browning's Patent Colourless Preservation Solution (The Indestructible Paint Co.), which is invisible when applied, is good for preserving sandstones and granites. It has been used on Cleopatra's Needle. The Silicate Paint Co., Charlton, supply a solution which is good for bricks and porous materials, but it is visible.

Pure baryta water, that is, a solution in water of barium hydrate, will penetrate into a decaying stone and the stone will become harder and more solid than when in its original condition. Professor Church applied this treatment to the Chapter House, Westminster Abbey.

This is only applicable in those cases where $CaSO_4$ has been formed. The action of $Ba(OH)_2$ is as follows :—

$$Ba(OH)_2 + CaSO_4 = BaSO_4 + Ca(OH)_2.$$

The $Ca(OH)_2$ is acted upon by the CO_2 of the air and reforms $CaCO_3$. $BaSO_4$ is insoluble and is not acted upon by the ordinary acids in the atmosphere.

Weight, Strength and Absorption.—The following is a

table giving the weight, strength and absorption of the stones in most general use :—

—	Weight per cubic feet in lbs.	Absorption in percentage of its dry weight.	Crushing load per square foot in Tons.
LIMESTONES—			
Ancaster Freestone	140·4	6·27	184·0
Box Ground	127·9	7·49	97·5
Coombe Down	128·6	5·80	117·7
Corsham Down...	129·0	11·06	94·5
Doulting Freestone	125·0	11·05	103·9
Ham Hill	136·0		166·3
Monks Park	136·7	7·74	139·6
Portland, Whitbed	132·3	7·51	204·7
DOLOMITES—			
Red Mansfield	143·2	4·58	591·9
White Mansfield	140·1	5·01	461·7
Yellow Magnesium Limestone	145·4	4·62	577·4
SANDSTONES—			
Blue Hailes	143·2	4·70	459·7
Bramley Fall	132·2	3·70	238·4
Corsehill	130·4	7·94	444·9
Craigleith	138·6	3·61	861·9
Dean Forest	151·4	2·71	530·0
Darley Top	139 0	3·40	516·7
Howley Park	140·3	4·90	466·7
Robin Hood	144·6	3·90	574·0
White Grinshill...	122·5	7·80	209·3
White Hailes	143·8	3·71	662·0
GRANITES—			
Aberdeen Corennie (Pink) ...	159·1	0·42	1318·3
,, Peterhead (Red) ...	158·5	0·29	1207·7
,, Rubislaw	163·7		1098·8
Cornish Grey	161·7		955·9

Artificial Stones—Classification.—The following classification has been based upon Howe :—

(1) Stones made of natural rock fragments held together by cement, such as Portland.

(2) Similar stones subjected to a subsequent hardening process by treatment with water glass, that is, silicate of soda.

(3) Stones in which rock has been pulverized or sand is cemented with carbonate of lime.

(4) Stones in which more or less of the carbonate of lime cement is replaced by silicate of lime.

(5) Stones cemented by bituminous, asphaltic or other organic substances.

Under the first heading would be included the ordinary concretes. The second heading includes those such as the "Victoria and Imperial Stones," the Hard York Non-Slip Stone, in which the action of the cement is modified and hydro-silicates are formed which harden the stone. The third heading would include those made under the Thom's patents, and known as reconstructed stone. The fourth heading comprises Ford's silicate of lime stone which is chemically formed; the fifth would include tar-macadam and such surface coverings.

Indurated Concretes or Artificial Stone.—Concrete slabs for paving, steps, sills, copings, dressings, pipes, etc., are now made by well-known firms, such as the "Victoria Stone" and the "Imperial Stone," by forming a concrete of about three of pulverized granite or sandstone aggregate with one of Portland cement. The artificial stone thus formed is found to be most durable in all positions, and is specially suitable for pavings and steps which have to resist a great amount of wear. The slabs or details are often cast in moulds placed upon rocking or trembling frames, to hasten the setting, then after about a day they are placed in tanks, and covered with sodium silicate (water glass), where they are left from 3 to 14 days, after which they are stacked in the open for about 6 months to mature.

The "Non-Slip Stone," made by pulverizing under great pressure the aggregate which is Silex Stone, then forming the cement concrete and subjecting to a great pressure of 2,000 tons or more to the square foot, and then treating by the hardening and maturing process, as already described for the Victorian Stone. Non-Slip Stone is eminently suitable for flagstones and all surfaces which have to resist footwear.

Reconstructed Stone.—This is the name given to artificial stones formed under the Thom's patent, which consists usually in taking the *débris* of limestone quarries, crushing it into grit, mixing with lime made from dolomite, heating

in a closed retort, probably up to 1,800° Fahr., which drives off the CO_2, and the calcium and magnesium oxides are reduced in a powdery state. The latter is then slaked, mixed with water, and consolidated under great pressure into blocks or moulds. At this stage the blocks are as soft as chalk. After exposure to the drying and hardening properties of the atmosphere it is capable of being sawn and worked by tools with ease and rapidity, the stone being without lamination or bedding and cleaving readily in any direction, and thus satisfying the conditions of a perfect freestone.

The blocks are now dried in a temperature of 100° to 120° Fahr., and placed in steel tanks, from which the air is exhausted. The CO_2 previously extracted is now admitted until the carbonization of the hydrate of lime blocks is complete.

Any kind of sedimentary rock, granular or metamorphic, can be reconstructed by this process. The Oolitic and Magnesian limestones and the Hopton Wood lend themselves most readily to this method of reconstruction.

Reconstructed stone is now being largely used and there is the probability of an extensive field open to its use for details, if it is used reinforced with steel.

Ford's Silicate of Lime Stone.—This artificial stone is formed of fine sand and chalk lime, in the ratio of 18 or 19 silica to 1 of lime, mixed and rammed dry into a perforated steel mould. A vacuum is formed in the mould when the lime is slaked with boiling water, as the water passes off through the perforations, superheated steam under a pressure of 120 lbs. to the square inch is applied to ensure thorough slaking and to form a silicate of lime. This produces a homogeneous sandstone, fine or coarse, and with a tint, varying with the aggregate, easy to work, less soluble in acids than most limestones and possessing a great resistance to crushing.

Bituminous Stone.—Diorite and other granite stones are often impregnated with prepared or refined tar, to form when laid a durable noise, wear and dust resisting stone surface.

CHAPTER V

BRICKS

Definition.—Bricks are an artificial kind of stone, made of burnt or baked argillaceous or clayey earth, and the quality of the bricks depends upon (*a*) the chemical properties of the earth, (*b*) the preparation of the earth, and (*c*) the different degrees of burning or baking.

Composition.—The following is approximately the chemical composition of a good brick-earth: Silica, three-fifths; alumina, one-fifth; oxides of iron, calcium, magnesium, manganese, sodium and potassium forming the remaining fifth.

Clay or aluminium silicate $(Al_2O_3)_x(SiO_2)_y(H_2O)_z$ forms the bulk of brick-earths. It possesses the property of plasticity when damp, but upon the application of sufficient heat it gives off its water, loses its plasticity, and becomes permanently rigid, and by no known process can its plasticity be restored. It contracts and warps during the process of burning.

Silica (SiO_2) is present, either chemically combined with alumina and water, or free in the form of flint and sand. Its presence in clays produces hardness, resistance to heat, durability, and prevents shrinkage and warping. An excess of silica causes bricks to be brittle.

Lime-stone or chalk ($CaCO_3$), when present in brick-earths, acts chemically in burning as a flux, causing the particles of the bricks to unite, producing greater molecular strength, and in small quantities diminishing contraction. An excess of calcium carbonate causes the bricks, in burning, to melt and lose their shape.

Magnesia (MgO) in the brick-earth influences the colour of bricks, tending to give a yellowish tint.

Iron influences the colour of bricks, but if occurring in clays, as iron pyrites (FeS_2), it should be carefully removed, otherwise it will oxidize in the brick, crystallize, and split it to pieces.

Brick-earths often contain various salts, as, for instance, those taken from the seashore or near salt formations contain a quantity of common salt; these render the brick-earths unfit for the manufacture of bricks. The salts in excess act as a flux in burning, causing the bricks to warp and twist, in addition to which these bricks, if exposed to the weather, absorb atmospheric moisture for a considerable time; the dampness causes efflorescence, which is very noticeable on new work. The salts, magnesium sulphate and calcium sulphate, if in the brick-earths or produced in the burning, will cause efflorescence: this can be chemically changed and the discoloration prevented by adding and mixing barium carbonate, usually about 1 oz. to every cwt. of brick-earth before the moulding process. This operation produces barium sulphate and magnesium carbonate or calcium carbonate, which are non-hygroscopic.

Analyses.—The analyses on the next page give an idea of the proportions of the chemical elements in some of the brick-earths. Nos. 1 to 7 are from Abney, No. 8 from Knap.

Very few brick-earths are in such condition as to allow of their being used without some special preparation. They are practically classified as plastic or strong clays, loamy clays, and marly clays.

Plastic or strong clays, known to the brickmaker as foul clays, contain silica, alumina, and but a very small proportion of foreign salts of lime, magnesia, and soda, and are sometimes described as pure clays. For the manufacture of bricks these clays require the addition of silica and lime.

Loamy or mild clays contain quantities of free silica, and are known as sandy clays. To these calcium carbonate is frequently added.

Marls or calcareous clays contain a large proportion of calcium carbonate, make good bricks, and are frequently used without the addition of other substances, but chalk

or sand is added if the natural earth is deficient in these compounds.

	1	2	3	4	5	6	7	8
	Fire-clays.		Ordinary Clays.					
	Dinas.	Stourbridge.	Loam.	Blue Clay.	Burham Clay.	Terra-Cotta Clay.	London Brick Clay.	Marl.
SiO_2	86·2	63·4	66·7	46·5	42·92	75·2	49·5	43·0
Al_2O_3	2·3	23·2	27·0	38·0	20·42	10·0	34·3	
Fe_2O_3	0·3	1·9	1·3	1·0	5·0	3·4	7·7	3·0
CaO	1·0	—	0·5	1·2	10·79	1·2	1·4	26·04
MgO	—	0·9	—	—	0·07	trace	5·1	3·5
Alkalies or Alkaline Chlorides	—	—	—	—	0·33	0·5	—	—
CO_2	—	—	—	—	8·12	—	—	20·46
H_2O	—	—	—	—	6·68	5·9	—	4·0
Organic Matter...	10·0	10·0	5·0	13·6	5·01	3·7	1·9	—
	99·8	99·4	100·5	100·3	99·34	99·9	99·9	100·0

Malm or washed earth is a prepared marl in which the quantities of the constituents are proportioned to give the best results, where bricks of a specially good quality are required. The brick-earth is ground to a pulp in a wash mill and mixed to the consistency of cream, with chalk previously ground. It is then passed through a screen or grid which excludes from the mixture any large particles or stones and ensures a fine division of the material, which is then conducted into settling tanks or pits. The particles are allowed to settle and most of the excess of water is run off, and a large portion of the remainder evaporates. The resulting pulp is known as malm. At this period the breeze necessary for the proper burning of the brick is spread over the compound.

Frequently malm is mixed with a proportion of ordinary unwashed brick-earth; the product is termed "malmed earth."

Test for Clays.—The brickmaking quality of a clay is usually ascertained by making a brick out of the clay in

question, and treating it, exactly as other bricks are treated, by firing it in a brick-kiln. If the brick does not come up to the required standard, chemical analysis will suggest what might be added to improve the earth.

The treatment of brick-earths varies in different brick-yards. The operations are in general as follows, and may be performed by hand or machine: first, the preparation of the brick-earth; secondly, the moulding; thirdly, the drying; fourthly, burning.

The following description of the manufacture of three classes of bricks will include the chief methods of brick-making, viz., (1) the hand-moulded clamp-burnt bricks, commonly known as stocks; (2) the machine-made wire-cut kiln-burnt; (3) the machine-made pressed, or hand-moulded, kiln-burnt bricks.

Hand-Moulded Clamp-Burnt.—For this method the earth is subjected to the following processes: Unsoiling; clay digging; stone picking, or washing and screening; addition of chalk, sand and breeze as required, and weathering; mixing; and tempering in a pug mill.

Unsoiling consists in removing the mould or top, which is often used for resoiling exhausted workings. The vegetable mould is known as Encallow, and the operation of removing as Encallowing.

Clay digging is usually performed in the autumn, when the clay is excavated and heaped up to the height of several feet on a levelled piece of ground prepared to receive it, any stones being carefully picked out by hand.

A layer of brick-earth is spread upon the ground, upon which is placed a layer of breeze, and then a layer of chalk, which latter has been previously broken up and mixed with water in a wash mill. This series of layers is repeated till the heap is 5 or 6 feet in height. It is then left through the winter months to be disintegrated and mellowed by the frosts.

An alternative process, where a better class of brick is required, is to wash the earth and chalk, if required, together in a wash-mill. The resulting compound, with a considerable quantity of water, is passed through a grid to ensure the particles being in a fine state of division; it is then

conducted to settling-pits, from which the excess water is removed. When the material is sufficiently firm, a layer of breeze to the required amount is spread over the top, and the whole left during the winter to weather.

In the spring the earth which has been left during the winter to weather is mixed, the heaps being cut in vertical sections to ensure the uniform distribution of the various materials throughout the mass. After being turned over two or three times, it is wheeled away in barrows to be tempered.

The object of tempering is to knead the earth into the proper condition for the moulding process. The usual method (when only comparatively small quantities are required) is to turn the clay over two or three times, kneading and battering it with shovels, and picking out any stones that may remain. Horses or men tread over the same, making the clay into a homogeneous mass. Where the demand for bricks is sufficiently great, the clay is tempered by being passed through a pug mill—a machine consisting of a circular stationary tub with a revolving vertical spindle, to which are keyed a number of knives, which, by their motion, cut, knead, and force the clay gradually through the pug mill, fitting it for the immediate use of the clot moulder.

Moulding.—The object of moulding, which is performed by hand, is to give the brick-clay a definite shape.

The operation of hand moulding consists in placing a wooden or iron box termed a mould, about 10″ × 5″ × 3″ (if the dimensions of the burnt brick are to be $8\frac{3}{4}$″ × $4\frac{1}{4}$″ × $2\frac{5}{8}$″, as the clay generally shrinks about one-tenth in all directions), without top or bottom, over a stock-board with a fillet or *kick*, about 7″ × 2″ × $\frac{3}{8}$″, fixed upon the same, and forming a projection upon the stock-board, which is secured to the moulder's bench. The mould is either (1) wetted or (2) sanded, so as to prevent the surface of the raw brick from adhering to its sides. The moulder then dashes and presses a clot of tempered clay, which he has immediately before kneaded with his hands, and from which he has removed any stones which may have escaped previous detection. He then takes the strike, which is

usually a pine fillet about 16″ × 1½″ × ⅜″, and draws and pushes off any superfluous clay over and above the level of the sides of the mould.

Drying.—Directly the clay has been moulded the operation of drying commences, the object being the evaporation of all superfluous moisture without damaging the brick—to render it sufficiently hard to be handled without injury, and to enable the raw brick to possess the requisite strength to withstand the pressure caused by stacking in the clamp during the process of burning.

When the method known as slop moulding is employed, the usual routine is for a boy to take the mould and moulded brick from the moulder, and place the raw brick on its bed upon a drying floor, which is slightly convex, and covered by a roof; the bricks are then sprinkled with sand to absorb superfluous moisture. After 1 day's exposure the raw bricks are placed upon their sides for another day, after which time they are sufficiently hard to be wheeled upon barrows to the hacks. The hacks are long parallel banks, usually 6 inches above the level of the ground, and built of brick rubbish and ashes, or sometimes of agricultural drain pipes at right angles to the length of the hack, and covered with a thin concrete bed, the object of which is to form a smooth horizontal bed thoroughly drained, so as to keep dry and to prevent the damp from rising.

If the method known as sand moulding be adopted, the moulder places a pallet (which is a piece of pine ⅜ inch thick, and about 1 inch wider and longer than an ordinary brick) upon the raw brick in the mould, then turns the whole over, releases the mould, and places the raw brick on the pallet upon a specially made wheelbarrow with springs, so as to reduce the vibration, which is dangerous to the raw bricks; when the barrow is loaded, the bricks are taken and hacked at once.

The bricks are hacked about ⅜ inch apart (the thickness of a pallet), being laid on a long narrow face and built about seven courses high, their ends exposed to the weather, their wide faces vertical and at right angles to the length of the hack. In that state they remain for about 10 days

after which they are scintled, that is, their wide faces arranged vertically and diagonally at an angle of 45 degrees to the length of the hack, with a space of about 2 inches between the bricks, the directions of the successive courses being reversed, so that the wind may get between and more effectually dry them, the whole operation of drying taking from 3 to 6 weeks.

During the time of drying, which takes place in the open air, the hacked bricks are protected from the weather by wood framing, covered with straw, matting, canvas screens, or tarpaulins.

Burning.—The object of burning is to drive the water from the clay and thus cause it to lose its plasticity, and to fuse the constituents into a homogeneous body, and to endow it with the necessary degree of hardness to resist compression for the purposes of building, and to vitrify it sufficiently to resist the disintegrating effects of the winter's frosts.

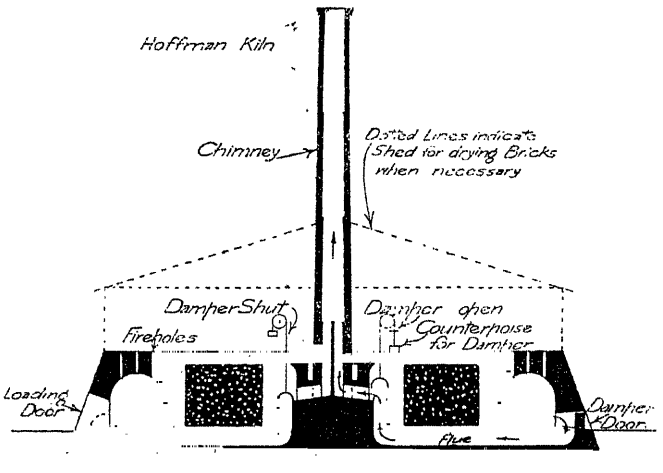
These bricks are burnt in clamps, the construction of the latter being as follows: The site is raised above the surrounding ground, and, to ensure dryness, is drained. This surface is paved with a layer of bricks (generally badly-burnt bricks from a previous burning), upon which a series of horizontal flues, termed fire-holes, are constructed; these flues are filled with faggots; over these two layers of bricks are laid on edge diagonally and about 2 inches apart, the interstices being lightly filled with breeze; over this a layer of raw bricks on edge is placed close together; over this is spread a layer of breeze, 7 inches in depth, then another course of raw bricks, on which is a second layer of breeze 4 inches in thickness; upon this another course of bricks on edge, then a layer of breeze 2 inches in depth. Above this the bricks are built in a series of bolts (that is, a thin unbonded wall) to a height of 14 feet. The time of burning is from 2 to 6 weeks according to the number of fire-holes and the atmospheric conditions.

The bricks produced by this method are termed stocks. These are generally employed for the internal parts of the walls of buildings, for which purpose they are eminently

adapted, being capable of resisting a great amount of compression, their surfaces forming a very effective key for plastering. Usually the better qualities of stocks are picked for facings, care being taken that as nearly as possible they should be of one tint, showing well-burnt faces.

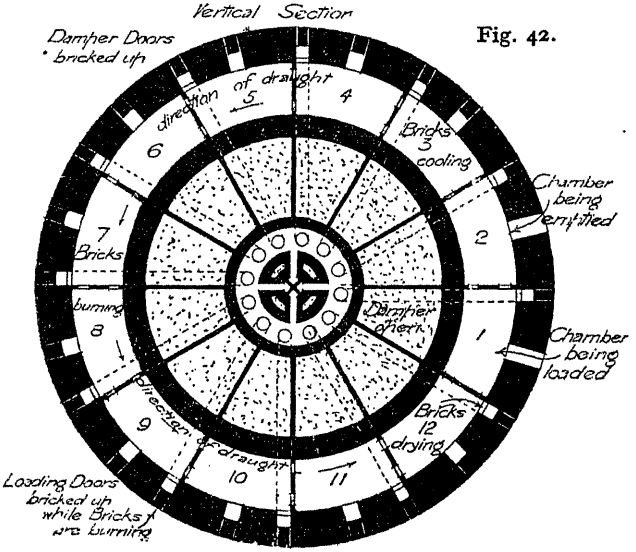
Scotch Kiln.—These kilns consist of four walls, usually without roofs. Fire-holes are arranged at the base, in which the coal for burning is added. These chambers are made sufficiently large to contain from 20,000 to 50,000 bricks. The bricks are stacked with a space between to allow the fire to permeate the mass. When the bricks are arranged the fires are applied gradually to drive off the moisture remaining in the bricks. This done, more fuel is applied, and the burning proceeded with. The top layer of bricks is protected by covering with old bricks to economize the heat. The bricks take from 2 to 3 days to burn, after which the fires are damped and the kilns allowed to cool gradually. Bricks burnt by this process are far more uniform in colour and regular in shape than clamp-burnt bricks. The best bricks are taken from the centre of the kiln ; the bottom layers are liable to be fused ; those at the top are generally underburnt, soft and unfit for face work. Kiln-burnt bricks may generally be classed as: Builders' first, from middle of kiln. Builders' second, from between first and third. Builders' third, bottoms and tops. The earth used is invariably a loamy clay containing a quantity of free silica, and generally the resultant colour is red.

Hoffmann Kiln.—The Hoffmann kiln (Figs. 42 and 43) is circular in plan and consists of an annular chamber divided by brick partitions with small openings at the bottom into twelve or more compartments, each of which is connected by a flue to a central chimney. The bricks as they are moulded are stacked into the compartments. Ten compartments would be full: Nos. 3 to 12, as shown in Fig. 43, No. 1 chamber would be in the process of loading and No. 2 of emptying. The flue in No. 12 compartment, leading to the central chimney, would be opened, the remainder of the flues are closed ; the draught is thus compelled to



Vertical Section

Fig. 42.



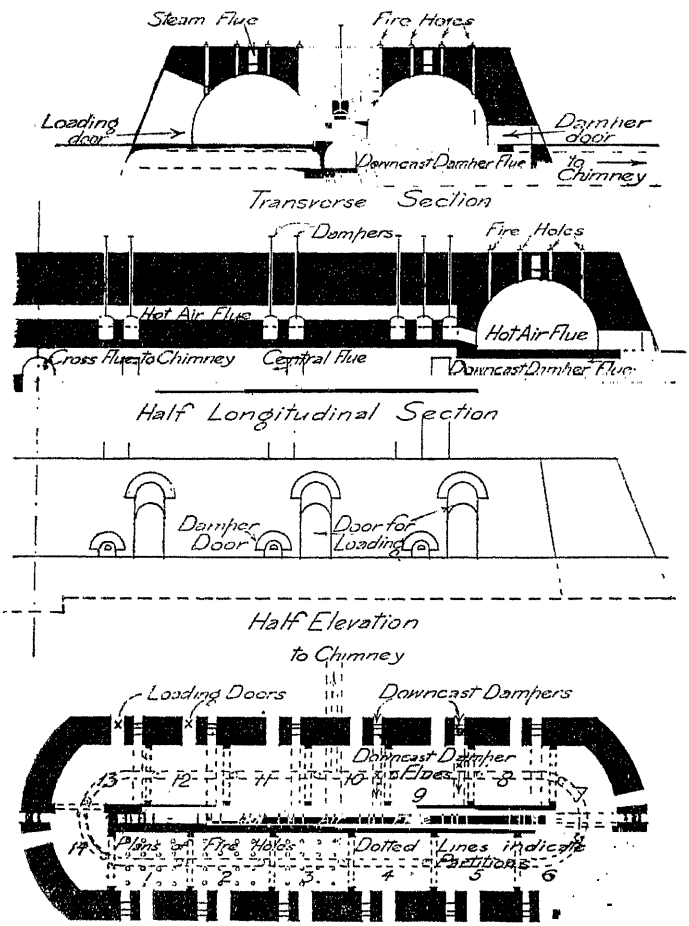
Plan

Fig. 43.

pass through the whole of the loaded compartments. The bricks in Nos. 3, 4, 5 and 6 would be burnt in the cooling stage, those in Nos. 7 and 8 would be in the process of burning and at their maximum temperature, those in 9, 10, 11, and 12 would be in varying stages of the drying process. The openings in the bottom of the partition between Nos. 12 and 1 would be covered by sheets of paper to prevent the draught passing through into No. 1 chamber, when the latter is loaded No. 2 will be empty, the openings at the bottom of the partition between Nos. 1 and 2 will be covered with paper. No. 1 flue will be opened and No. 12 closed. The draught would now be sufficient to remove the paper dampers between Nos. 12 and 1. The cycle of operations may be repeated without interruption until the kiln is stopped for repairs. After each compartment is charged with bricks, the loading door is bricked up.

Improved Hoffmann Kiln.—There are various modifications of the original Hoffmann kiln, one of the latest being Warren's patent "Perfected kiln." This is an application of the regenerative principle to the burning of bricks. It is rectangular in plan, the ends at times being rounded; it has a number of chambers, usually 14; the divisions between the chambers consist of brick partitions. A space adjacent to the divisions is left for the fuel, and the remainder of the chamber is filled with bricks closely stacked. Each chamber has an upcast flue to carry off steam from the top of the chamber during the drying period. A down-draught flue for the purpose of carrying off steam and products of combustion from the lower part of the chamber is placed in these kilns near the outside wall, and is regulated by a damper from the outside. In addition to these flues there is a central hot air flue to which each chamber is connected. The mode of firing and burning is similar to that described for the ordinary Hoffmann kiln, but in these the cooling process is greatly facilitated and the heat considerably economized by the use of the central hot air flue. In the chambers that are cooling the loading door is opened, also the duct leading to the central hot air flue. The duct leading from the hot air flue to the chamber containing bricks in the drying stage is opened; this causes a draught

Warren's Perfected (Warren's Patent) or Improved Hoffmann Kiln



Figs. 44–47.

from the cooling chamber through the cooling bricks, through the hot air flue, and thence through the drying bricks and finally through the downcast flue and away up the chimney stack; this is the adaptation of the regenerative principle, and constitutes the advantage of this system. Figs. 44–47 illustrate this form of kiln, which may be advantageously used in drying and burning for face as well as ordinary bricks.

Stocks are classified according to their quality as follows: Malms, Malmed, and Common.

Malms.—Cutters, Best Seconds, Mean Seconds, Pale Seconds, Brown Facing Paviors, Hard Paviors, Shippers, Bright Stocks, Grizzles, Place.

Malmed.—Bright Fronts, Stocks, Shippers, Hard Stocks, Grizzles, Place.

Common.—Stocks, Shippers, Grizzles, Rough Stocks, Place, manufactured from unwashed earth.

1. *Cutters or Rubbers.*—Made from washed earth containing sufficient sand necessary for a burnt brick which is required to be easily divided with a brick-cutter's saw. The best are burnt to a state little short of vitrification.

2. *Seconds.*—Similar to No. 1, but uneven in colour.

3. *Facing Paviors.*—Hard-burnt malms of good shape and colour, used for facings of superior walls.

4. *Bright Fronts.*—Similar quality from malmed earth.

5. *Hard Paviors.*—More burnt; slightly blemished in colour; used for superior paving, coping, etc.

6. *Shippers.*—Sound hard-burnt bricks, imperfect in form; used as ballast for ships.

7. *Stocks.*—Hard, sound, fairly uniform in colour; they are used for the mass of ordinary good work.

8. *Hard Stocks.*—Overburnt, but sound; slightly misshapen, and colour not uniform; they are used in footings and in the body of thick walls, and in positions where the work is subjected to a great compressional stress.

9. *Grizzle.*—Underburnt, but sound and of good form; used for inferior or temporary work, and where not subjected to heavy loads.

10. *Place.*—Underburnt, weak; containing stones, causing them to be very liable to breakage; for inferior or temporary work. Sometimes place bricks are used in the panels of brick-nogged partitions for the purpose of retarding sound.

11. *Chuffs.*—The action of wind, frost or rain upon bricks while hot, on the outside of clamps, or if the bricks are put into clamps before they are sufficiently dried, causes the bricks to be full of cracks and useless for constructional purposes; such bricks are termed chuffs.

12. *Burrs.*—Lumps of bricks vitrified and run together. They are useful for rough walling, artificial rock work, etc.

Stock Bricks, Machine-Made.—Messrs. Eastwood & Co., of Conyer, Kent, make use of a patent method for the manufacture of stock bricks by machinery with patent drying chambers and kiln. Great care is taken in the proper admixture of the usual materials, which is run with a large proportion of water into settling pits. After the surplus water has been drained off and the prepared earth is sufficiently mellowed by weathering, it is then barrowed into the pug mill, and thence passed to the moulding machine below, where it is pressed into a mould containing six bricks. Upon removal from the moulds, the raw bricks are placed upon specially-constructed trolleys working upon tram-lines, and conveyed to the drier. This consists of three long chambers, through which the heat is regulated by means of fans, the temperature varying from 45° to 200° Fahr. The bricks by this method are fit for the kiln in 24 hours. For burning, they are then stacked

upon specially-constructed trolleys running upon a tramway. The upper portions of the trolleys for a considerable thickness are formed of fire-brick. These are passed into one end of the kiln, which is a chamber 180 feet long, only slightly larger in section than a loaded truck. It emerges after 3 days, after having passed through a heat gradually increasing in intensity towards the centre of the kiln and then decreasing. Fuel is supplied through fire-holes in the roof of the kiln. The resultant is bricks more uniform in shape and colour than the ordinary hand-moulded brick, and, in addition, these are turned out at a much greater rate than by the hand process, and the manufacture can be carried on throughout the whole year. There is also great economy, as it is anticipated there will not be more than from 3 to 10 per cent. of grizzles or waste.

Kiln-Burnt Red Bricks, Hand-Moulded.—The processes through which this brick passes are as follows: Clay getting, washing, weathering, pugging, moulding, drying, burning. All these processes but the burning are carried out as previously described for malm or washed earth bricks. When the bricks are sufficiently dried they are burned in a kiln.

Machine-Moulded Wire Cuts.—The operations to produce these bricks are as follows: Clay getting, stone picking if necessary, grinding, or weathering and grinding, pugging, pressing, and squeezing through an orifice in a strip of about $4\frac{1}{2}'' \times 9''$ section. It is then cut into 3-inch layers by means of wires arranged in a frame. The usual method of burning these bricks is in a Hoffmann or similar kiln.

Machine-Made Fletton Bricks.—Of late years large quantities of these bricks have been made in the neighbourhood of Peterborough, the output being at the rate of 8,000,000 per week.

The operations consist of clay getting, drying, grinding, sifting, pressing and burning.

The clay, which is obtained from the Oxford clay formation, is a dense bluish-grey shale, is dug, if necessary dried on a drying floor to expel superfluous moisture, ground

in a mill similar to a mortar mill, the receiver revolving and carrying the clay under the rollers. The bottom of the receiver is perforated, which allows the material, when sufficiently ground, to fall through as a coarse powder into a pit beneath. It is then elevated and shot into a revolving circular inclined sieve; the material that is sufficiently fine passes through, the remainder is conveyed back to the grinding mill. The sifted powder falls into a hopper, whence it is passed to the pressing machine, where by an ingenious arrangement it is measured off and passed under the die, where it is pressed into the form of a brick, and removed automatically from the mould. The bricks are then placed on barrows and stacked in the kiln, which latter is of the Hoffmann type, and then burnt. The time of digging the clay to the stacking in the kiln need not occupy more than 15 minutes, the time taken to burn the bricks usually taking about 3 weeks. These bricks are of a good form, compact, and are useful for all internal work. Their colour, a reddish-yellow, is not such as would fit them for the best facing work. For internal work to receive plaster, special bricks are prepared with dovetailed-shaped grooves.

The average Fletton brick when immersed in water for 24 hours will absorb 20 per cent. of its weight of water.

The usual dimensions of Fletton bricks are $8\frac{3}{4}'' \times 4\frac{1}{4}'' \times 2\frac{5}{8}''$, weigh about 5·6 lbs. each, or, say, $2\frac{1}{4}$ tons per 1,000, but there is unfortunately no standard, and many different sizes are made.

The composition of the "Knotts" clay from which Fletton bricks are made is more or less as follows :—

Silica	50 per cent.
Alumina	16 ,,
Ferric Oxide	7 ,,
Calcium Carbonate	10 ,,
Magnesia	1 ,,
Alkalies	3 ,,
Organic Matter	6 ,,
Water	7 ,,
	100

but varies with the locality and with the depth below the surface from which the clay is taken; the carbonate of lime

is chiefly due to fossils, the quantity of which is very variable.

Indications of Processes.—The following hints are useful in determining some of the processes which a finished brick has passed through.

Hand-Moulded.—Frog on one side ; and no great amount of finish in the form, and porous.

Wire Cuts.—No frogs ; wire marks on the beds ; regular in form and dense.

Pressed Bricks.—These have some or all smooth faces, sharp and regular arrises, clean frog or frog on both sides, and trade marks in the frog. These bricks are very dense.

Clamp-Burnt.—Colour (of the bricks) is not uniform ; the traces of the breeze can be seen, especially if the bricks are broken across, when also the internal colour will be noticed to be darker and the texture slightly vitreous.

Kiln-Burnt.—Light and dark stripes upon the sides caused by the bricks being arranged with intervals between them while burning, the exposed parts being burnt to a light colour, and where resting upon or against another brick to a dark colour. This may be remedied by stacking the bricks upon their front faces in contact while burning.

Colour.—The colour of bricks is affected :—

(*a*) By the chemical constituents of the brick-earth.

(*b*) By the sand which has been sprinkled upon the raw bricks before being burnt.

(*c*) By the degrees of heat to which a brick has been subjected during the process of burning.

(*a*) The colour is determined chiefly by the quantity of iron present in the clay.

Bricks manufactured from clay free from iron burn white, and such clays, containing but a small quantity of chalk, together with iron, give a cream colour.

With a small quantity of chalk, but additional iron, a

red colour is produced, and an additional quantity of chalk gives a brown.

Clays possessing from 8 to 10 per cent. of iron give in burning a blue or almost black colour.

Bricks in burning are exposed to a great heat, and if the clay contain alkalies, and be burnt at a still higher temperature, a bluish-green is produced, as in the case of Staffordshire bricks, which, under ordinary circumstances, are red.

White bricks usually contain but the merest trace of iron.

Blue bricks are prepared from earth containing a large proportion of oxide of iron.

Black bricks are made from a similar clay to the blue bricks, but in addition the earth contains a small quantity of manganese.

To obtain a clear bright red brick the clay should be free from impurities, and contain a large quantity of oxide of iron, which by burning is converted into the red oxide, but not fused.

Magnesia in the presence of iron makes the brick yellow.

In clamp-burnt bricks the sulphur contained in the ashes gives them a yellow or brimstone tint.

(*b*) The sand sprinkled upon the raw bricks before they are burnt is vitrified in burning, and this will to a great extent affect the surface colour, which is but skin deep.

(*c*) As a general rule, the greater the amount of heat to which a brick has been subjected in burning the darker the tint.

The varieties of bricks in common use in the neighbourhood of London include :—

Stocks, Fletton, Wire Cuts, Gault, Red, Blue, Black, Paviors, Fire Bricks, Salt-glazed, Enamelled.

The first three have been previously described.

Gault Bricks.—These are made from a bluish clay interposed between the upper and the lower greensand. The composition of the Gault clay varies. These bricks are made by the Barham Company, near Rochester, also at Hitchin. Although of a fairly uniform dark blue colour, the clay contains at times (comparatively speaking) large

quantities of the hydrous oxide of iron ; at others, a good deal of calcium carbonate is found in combination.

The bricks are all burnt in the kiln, in the former case into a deep red brick or tile of an inferior quality ; in the latter, perforated, hard, white bricks are made.

These bricks require great care in burning, for if the calcination of the calcium carbonate takes place under such conditions that the lime is left in a caustic state, it will slack on exposure to the weather, or when moisture is applied to it.

Suffolk Bricks, called also *White Suffolks*.—These are Gault bricks, and are kiln-burnt, being expressly made for facings, but they are expensive. The best can rarely be obtained in London, being sold in the locality of their manufacture. They have a disagreeable cold hue, rendered still more dull after a few years' wear in a smoky atmosphere. They are not as well burnt as those possessing a somewhat light pink or salmon tint.

Beart's Patent Bricks.—These are Gault bricks, made at Arlesey, near Hitchin on the L. & N. E. Railway, and comprise the following, ranged according to price :—

(*a*) White Rubbers, which are hand-made, moulded, solid, and equal to the best Suffolks.

(*b*) No. 1, best selected, white facing, pierced brick, are of uniform colour, hard, well burnt, and extensively used for facings.

No. 2, red and pink blended, differing from No. 1 in colour only, and in every way equal to the best made stock bricks.

Red Bricks.—These bricks are made from a loamy clay in which the sand contains a considerable quantity of iron. These bricks are used as facings. A class of these bricks, prepared from brick-earth upon which a special treatment of careful washing, weathering and tempering has been carried out, and which contains an excess of sand above that usually employed, are termed cutters or rubbers. They can be easily cut to any required shape by means of a brick saw, and in consequence of their

fine texture can be rubbed to sharp arrises, and are also suitable for carving. They are largely used for arches and decorative work. The "Fareham Reds," and those supplied by Messrs. Blanchard, of Bishop's Waltham, are noted bricks of this class; and also the well-known T.L.B. rubbers, made by Messrs. Lawrence & Co., Bracknell, which are supplied largely to the London market.

Blue bricks are made from clays containing about 7 to 10 per cent. of oxide of iron. Large quantities of these are made in Staffordshire from the clays in that district. They are either wire cut or pressed, of a very dark blue colour, highly vitrified, very hard, dense, and capable of resisting great pressures. The same clay, if less burnt, produces a red brick.

They are largely used for engineering works and for piers where great compressional resistance is required. Special bricks are pressed for coping, channels and paviors.

Black bricks are made from earths containing a large proportion of oxide of iron together with oxide of manganese and burnt at a high temperature, and are especially useful for polychromatic work.

Paviors.—The following are used for the purposes of paving: Hard stock paviors, Blue Staffords, Dutch clinkers, and Adamantine clinkers. The first two have already been mentioned.

Dutch Clinkers.—These are very small, kiln-burnt at a high temperature, hard, are used for paving, and usually made 6″ × 3″ × 1″, are vitrified throughout, and sometimes warped.

Adamantine Clinkers.—These are bricks similar to the above, but harder, denser, and heavier. They are of a fine pink-white colour, and present a smooth surface. The edges are sometimes chamfered in order to give a firmer foothold when used for paving. They are made of numerous sections, such as kerbs, channels, etc., and of varying dimensions.

Fire Bricks.—These are so named on account of the resistance they offer to high temperatures. All bricks are

to a greater or less extent heat-resisting, but those having a fusing-point under 2,000° Fahr. would not be classed as fire bricks. Good fire bricks should show no signs of fusion when subjected to 2,876° Fahr., while those that show no signs of fusion when subject to a temperature of 3,038° Fahr. would be classed as first-class fire bricks. The resistance depends chiefly upon the relative quantities of silica, alumina, and oxide of iron present in the clay. They are yellow in colour, close in texture, and are made to the dimensions of an ordinary brick. The loam of which they are made is of a yellow colour, rough to the touch, and contains a considerable quantity of sand.

Good fire bricks should be thoroughly dried, and then evenly burnt throughout at a temperature of not less than 2,500° Fahr. for a period of from 12 to 14 days. They should contain no holes or flaws, and the surfaces should be free from flaws or winding. A test piece, when heated to a temperature of 2,462° Fahr., should not show more than 1 per cent. of linear contraction or expansion.

It is the custom now at some works to supply fire-brick material of a chemical analysis most suited to the purpose for which the bricks are to be used, such analysis varying chiefly in the ratio of silica to aluminium contained in the clay. This is done by Messrs. Williamson, Cliff Ltd., of Stamford, who own three good seams of fire-clay in the locality, but who, following modern German practice, import many other fire-clays regardless of distance or cost. The bricks are highly serviceable for the lining of furnaces and ovens. They are made in various parts of Wales and called "Welsh Lumps." Fire-clay is found and worked in various parts of the British Isles, at Wortley, in the west of Scotland, in Wales, Newcastle, Stamford in Lincolnshire, Poole in Dorsetshire, and at the Hurlford works near Glasgow. Perhaps the best-known beds are at Stourbridge, which supplies the London market chiefly, but the material is dear. The Dinas brick, manufactured by the Ynysmudw Company, near Swansea, is said to resist a heat greater than the Stourbridge brick.

Salt-glazed Bricks.—These bricks have a thin glaze on their exposed surface caused by throwing salt in the kiln

fire during the process of burning. This is explained in the article on stoneware.

Enamelled Bricks.—These have a white, light yellow, or other coloured surface like that of china, this surface being produced by covering a partially-burnt brick with a thin coating of white or other enamel over the required surface, and then reburning the brick. This is known as biscuit ware. The vitreous base of most enamels is a lead glass, but this base is objected to as being dangerous for the workers. In modern work leadless glazes are often specified, the lead base being displaced by such as sodium, potassium, zinc, tin or other metallic oxide, according to the tint required. Enamelled bricks of various colours, including white, are now manufactured by enamelling the raw brick and fixing the tint in one burning. These are more durable than those made by the biscuit process, but the best bricks of this make are expensive, as many are spoilt in burning. These bricks are much used for the sake of cleanliness in lavatories, dairies, etc., and also to reflect light in contracted areas.

Opalite.—This is a vitreous compound made in thin sheets of $\frac{1}{8}$ inch bare in thickness, the front presenting a highly glazed appearance, the back being covered with rough particles of the same material, burnt on to form a key. This material is made of varying dimensions, frequently of sizes of the external faces of bricks, to which they are attached by being bedded on a plaster specially prepared by the patentees. It is prepared in various colours, and is extensively and successfully used at a much less cost for all purposes for which glazed bricks would be suitable.

Crystopal.—Crystopal is a vitreous opaque compound, manufactured in thin sheets about $\frac{1}{8}$ inch in thickness, with a highly glazed surface, prepared in different colours, and similar in essentials to opalite, the chief difference being the key, which with this material consists of a mastic of an elastic nature; it is applied to surfaces of brick, stone, or concrete, which are rendered in cement to form a ground;

also to wood surfaces, to which it is secured with a mastic. It is claimed for this material that it does not crack through the differences in expansion and contraction of the different materials, the elasticity of the mastic compensating for these differences.

Characteristics of, and Tests for good Bricks.—Regularity of shape, uniformity of size, rectangular faces—only one end and side need be smooth—of uniform texture, compact and free from flaws of every description and of a good colour if stocks of a golden yellow capable of reflecting in a manner pleasing to the senses the golden rays of the sun. The quantity of water absorbed by a brick is a good test of its quality. When saturated they should not absorb more than about 15 per cent. of their own weight in water, they should absorb it reluctantly, and part with it freely at moderate temperatures.

They should be uniformly burnt, hard, and give a metallic ring when two are knocked together—a dull sound indicating a soft or shaky brick; should be of a good colour for their kind, sound when broken, tough or pasty in texture, not granular; should require repeated blows before breaking rather than one hard blow; should stand cartage and handling well.

Absorption.—Insufficiently burnt bricks absorb a large quantity of water and are not durable.

The absorption of bricks varies from one-fifth to one-fifteenth of their weight.

Machinery.—In the manufacture of bricks, where a great quantity is required, and where there seems a possibility of a regular demand, machinery is largely employed.

By machinery they are produced more cheaply, less labour being required.

Machines used for tempering the clay are called Pug Mills. Grinding machines consist of a pair of horizontal rollers, for crushing small fragments remaining in the clay. If, in addition, the clay is forced through an orifice, and then cut by wire according to the dimensions required, they are known as wire-cutting machines.

Still more recently machines have been used for tempering, moulding and pressing into the required form at one operation; that is to say, that the raw clay is placed in the machine, and is taken from it as a pressed brick.

Bricks, after the hand-moulding process, are often placed in a hand-pressing machine, with the object of correcting the form, and producing smooth faces and sharp regular arrises.

Strength of Bricks.—Bricks are subjected in practice to compression, sometimes to transverse stress, but not to tension, except such as would be caused by wind pressure or other lateral forces.

Considerable difficulty exists in the determination of the absolute strengths of bricks, owing to the variable nature of bricks even of the same type. In practice it is required to know the strength of brickwork in piers and walls. While it is comparatively easy to obtain the strength of individual bricks, it is not practicable under working conditions to obtain the strength when built into a wall or pier, owing to the great expense involved.

Exhaustive tests have been made by the Department of Scientific and Industrial Research, the results of which are embodied in a special report.

The following table gives the mean of results of a large number of tests on a series of well-known bricks and of piers built from them, also the strength of the mortar employed, with the ratio of the strength of piers to that of the bricks. The ratio is affected by the mortar employed, and it is noticeable that with bricks of a high resistance the ratio of the pier to the bricks is smaller than that where the bricks have a lower resistance, due to the great discrepancy between the strength of the mortar and the bricks.

For the further data on other physical properties and tests, see the special report of the above Department.

The British Standard Specification No. 449 gives the permissible values for brickwork built from bricks which have been graded according to the strengths of individual bricks. The factor of safety taken being approximately one-tenth.

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Influence of Strength of Bricks on Strength of Brick Piers

Pier Test No.	Description of bricks	Tests on materials: Bricks (see Table 1) Crushing strength (between plywood allowing for frogs) Lb. per sq. in.	Bricks: Modulus of elasticity Lb. per sq. in.	Mortar: Crushing strength Lb. per sq. in.	Mortar: Tensile strength Lb. per sq. in.	Mortar: Modulus of elasticity Lb. per sq. in.	Tests on Piers: Actual dimensions of pier	Overall strain at fracture In.	Crushing load* Tons	Crushing stress Lb. per sq. in.	Modulus of elasticity at load of 50 tons Lb. per sq. in.	Ratio Pier strength / Brick strength
I	Leicester Red Wire-Cut	3,920	$1{\cdot}22 \times 10^6$	2,610	312	—	$18\frac{1}{2}'' \times 18\frac{1}{2}'' \times 8'$	0·134	140	920	$0{\cdot}98 \times 10^6$	0·23
II	Fletton (*a*)	2,520	$0{\cdot}82 \times 10^6$	2,685	297	—	$18\frac{3}{8}'' \times 18\frac{3}{8}'' \times 8'$	0·214	127	842	$0{\cdot}69 \times 10^6$	0·33
III	Blue Stafford, Pressed	16,600	$4{\cdot}80 \times 10^6$	2,740	358	—	$18\frac{1}{2}'' \times 18\frac{1}{2}'' \times 8'$	0·143	473 5 (345)	3,180	$2{\cdot}49 \times 10^6$	0·19
IV	Sand Lime (*a*)	5,370	$2{\cdot}14 \times 10^6$	3,180	337	—	$18\frac{3}{8}'' \times 18\frac{3}{8}'' \times 8'$	0·183	264	1,750	$1{\cdot}31 \times 10^6$	0·33
V	Eastwood Stocks	3,420	$0{\cdot}90 \times 10^6$	2,920	315	—	$18\frac{3}{8}'' \times 18\frac{3}{8}'' \times 8'$	0·183	170 (155)	1,130	$0{\cdot}94 \times 10^6$	0·33
VI	Fletton (*a*)	2,520	$0{\cdot}82 \times 10^6$	3,020	363	—	$18\frac{1}{2}'' \times 18\frac{1}{2}'' \times 8'$	0·198	134 (125)	877	$0{\cdot}74 \times 10^6$	0·35
VII	Southwater, Pressed	5,230	$5{\cdot}04 \times 10^6$	3,210	399	—	$18\frac{1}{8}'' \times 18\frac{1}{8}'' \times 8'$	0·108	320 (310)	2,180	$2{\cdot}34 \times 10^6$	0·42
VIII	Accrington Ordinary Red, Pressed, with Two Frogs‡	10,300	$5{\cdot}98 \times 10^6$	2,750	455	$3{\cdot}06 \times 10^6$	$18\frac{1}{2}'' \times 18\frac{1}{2}'' \times 8'$	0·100	365 5 (320)	2,460	$3{\cdot}59 \times 10^6$	0·24
IX	Phorpres Fletton	4,700	$1{\cdot}43 \times 10^6$	2,570	472	—	$18\frac{1}{8}'' \times 18\frac{1}{8}'' \times 8'$	0·185	215 (210)	1,465	$1{\cdot}21 \times 10^6$	0·31
X	Stourbridge Glazed	2,930	$0{\cdot}97 \times 10^6$	2,280	350	—	$18'' \times 18'' \times 8'$	0·207	180 (140)	1,245	$0{\cdot}98 \times 10^6$	0·43
XI	Arlesley Mingled	3,000	$0{\cdot}67 \times 10^6$	3,010	436	—	$18'' \times 18'' \times 8'$	0·147	149 (140)	1,030	$0{\cdot}90 \times 10^6$	0·34
XII	Fletton (*b*)	2,480	$0{\cdot}79 \times 10^6$	2,960	396	—	$18'' \times 18'' \times 8'$	0·219	155 (120)	1,070	$0{\cdot}83 \times 10^6$	0·43
XV	Tunbridge Wells Wire-Cut Engineering‡	8,920	$1{\cdot}97 \times 10^6$	2,970	319	$2{\cdot}97 \times 10^6$	$18\frac{1}{2}'' \times 18\frac{1}{2}'' \times 8'$	0·206	400·5 (395)	2,620	$1{\cdot}39 \times 10^6$	0·29
XVI	Cellular Fletton	1,870	—	2,870	304	$2{\cdot}43 \times 10^6$	$17\frac{5}{8}'' \times 17\frac{5}{8}'' \times 8'$	0·140	117·5 (105)	850	$0{\cdot}76 \times 10^6$	0·45
XVIII	Red Hand-made Facing	1,660	$0{\cdot}60 \times 10^6$	2,670	297	$2{\cdot}25 \times 10^6$	$18\frac{1}{4}'' \times 18\frac{1}{4}'' \times 8'$	0·324	125 (110)	842	$0{\cdot}29 \times 10^6$	0·51
XIX	Sitting First Hard Stocks	1,850	$0{\cdot}75 \times 10^6$	3,000	185	$2{\cdot}42 \times 10^6$	$18'' \times 18'' \times 8'$	0·223	201 (175)†	1,385	$0{\cdot}84 \times 10^6$	—
XX	Do.	1,850	$1{\cdot}11 \times 10^6$	3,960	422	$2{\cdot}61 \times 10^6$	$18\frac{3}{4}'' \times 18\frac{5}{8}'' \times 8'$	0·190	125 (120)	803	$0{\cdot}60 \times 10^6$	0·43
XXI	Cattybrook Best Blue Wire-Cut‡	13,820	$4{\cdot}88 \times 10^6$	3,770	399	$2{\cdot}51 \times 10^6$	$18\frac{1}{4}'' \times 18\frac{1}{4}'' \times 8'$	0·092	422 (350)	2,840	$3{\cdot}51 \times 10^6$	0·21
XXII	Cattybrook Best Blue Pressed‡	7,680	$4{\cdot}09 \times 10^6$	3,830	405	$2{\cdot}70 \times 10^6$	$18\frac{1}{8}'' \times 18\frac{1}{8}'' \times 8'$	0·097	375 (330)	2,560	$2{\cdot}66 \times 10^6$	0·33
XXV	Sand-Lime (*b*)	3,060	$0{\cdot}93 \times 10^6$	2,290	217	$2{\cdot}40 \times 10^6$	$18'' \times 18'' \times 8'$	0·171	140 (110)	967	$0{\cdot}61 \times 10^6$	0·32
XXVI	Sand-Lime (*c*)	5,500	$1{\cdot}77 \times 10^6$	1,900	146	$2{\cdot}20 \times 10^6$	$18'' \times 18'' \times 8'$	0·150	218 (205)	1,500	$0{\cdot}93 \times 10^6$	0·27
XXVII	Aylesford Pink Wire-Cut	2,510	$1{\cdot}14 \times 10^6$	2,330	216	$2{\cdot}42 \times 10^6$	$18\frac{1}{4}'' \times 18\frac{1}{4}'' \times 8'$	0·139	119 (105)	800	$0{\cdot}73 \times 10^6$	0·32
XXVIII	Aylesford Perforated Wire-Cut	2,580	$1{\cdot}32 \times 10^6$	2,000	184	$2{\cdot}20 \times 10^6$	$18\frac{1}{8}'' \times 18\frac{1}{8}'' \times 8'$	0·108	144 (95)	985	$1{\cdot}02 \times 10^6$	0·38
XXIX	Sand-Lime (*b*)	3,080	$0{\cdot}93 \times 10^6$	2,670	194	$2{\cdot}45 \times 10^6$	$18'' \times 18'' \times 8'$	0·179	145 (140)	1,000	$0{\cdot}72 \times 10^6$	0·33

* Figures in brackets are Loads at First Crack. † Picked bricks only in pier. ‡ Bricks 3″ nominal depth. See Appendix II

[*To face p.* 136.

CHAPTER VI

TILES, TERRA-COTTA AND STONEWARE

TILES

Definition.—Tiles are thin slabs of brick-earth, burnt in kilns used for covering roofs, paving, etc.

Preparation.—The clay is prepared in a similar manner to that of bricks, but all the operations are conducted with greater care, especially in separating all the stones, on account of the thickness required being so small.

Shape.—Tiles for covering roofs are made in many forms and patterns; two of the most generally known and used being the plain and pan tiles.

Plain Tile.—The plain tile is rectangular in shape, the dimensions being $10\frac{1}{2}'' \times 6\frac{1}{2}'' \times \frac{1}{2}''$ in thickness. These tiles are made with two holes, through which are driven oak pins to hang over the laths, or nails to fix to the boarding or laths. The tiles are now often made with small projecting nibs on the top under-edge to hang to the laths, every fifth course being nailed; when hung on steep slopes or vertical faces, they are all nailed.

The tiles are made slightly rounded in their length, the concave surface being kept under, in order that the bottom edge may bite well on to the tile below.

Plain tiles are made in three widths—half tile, tile, and tile and half, the first being used as the end tile of every alternate course to break joint. In many cases where secret gutters are used, or in similar positions, there is a difficulty in fixing a half tile properly; tile and half tiles are substituted to ensure an efficient fixing.

Special tiles are made to cover hips and valleys, and also to cover vertical angles where walls are weather tiled. Ridge tiles are made either plain or to ornamental patterns, special pieces being moulded for stopped ends, hipped ends, and intersecting ridges, or ridges intersecting with slopes.

Pan Tiles.—Pan tiles are made of a flat S shape 14″ × 9″ × ½″; they are originally moulded flat, being bent to shape afterwards; they have a projecting nib on the underside of the top edge, by which they are hung to the battens. The hips and ridges are covered with segmental tiles bedded in mortar. These tiles when laid have a lap of 3 inches over the head of the tile immediately below; they do not lie closely together, and for that reason have to be bedded in mortar. Pan tiles are more efficient than plain tiles for flat pitches.

Paving Tiles.—Paving tiles are usually thicker than roofing tiles, varying from ½ to 1¼ inch, some being made from a similar clay to that for ordinary tiles; others are mixed with various substances to colour them. These are made in squares, hexagons, and other geometrical patterns.

Manufacture.—The tiles are manufactured in a similar manner to bricks, going through the processes of clay-getting, tempering, moulding, drying, after which they are beaten to correct their shape, and then baked in a kiln.

TERRA-COTTA

Terra-cotta is a material made from a refractory brick-earth, carefully selected and prepared; it is usually moulded into blocks, and built to represent ashlar work, being made up to 18 inches in length. The size of blocks of terra-cotta should never be made to exceed 4 cubic feet; undoubtedly blocks the contents of which do not exceed a cubic foot are much to be preferred.

Composition.—The following shows the composition of a typical specimen of terra-cotta :—

Silica	75·2
Alumina	10·0
Ferric Oxide	3·4
Calcium Oxide	1·2
Magnesium Oxide	trace
Alkalies and Alkaline Chlorides	0·5
Water	5·9
Organic Matter	7·7

It will be seen by the large amount of silica and the small percentage of alkaline matter present that the clay will be refractory, the quantity of the latter being just sufficient to cause a vitrified skin or surface, to which the material owes its durable qualities. The quantities of oxide of iron present, varying from the above amount to 10 per cent., impart the shades of pink and red common to terra-cotta.

Manufacture.—The material is carefully ground, strained and pugged, and, to avoid excessive shrinkage in drying, sand, ground glass, or pottery is sometimes added. The mixing must be conducted with great care in order to ensure uniformity throughout the mass.

The material is moulded, carefully dried, and then baked. The latter two operations must not be accomplished in a hurried manner, or the material will twist.

The terra-cotta blocks are not made solid, but are built up hollow, the thickness of the sides varying from 1 to 2 inches, and have diaphragms or partitions connecting the opposite sides for their support. By this arrangement great thicknesses of the material are avoided, the drying is facilitated, and the surfaces and the interior are more uniform, thereby avoiding fractures, which would otherwise ensue

In moulding the blocks, the thickness of the sides should be made of the same material throughout, and not made in two layers, as has been done, to economize the fine clay by placing a thin layer in front and using a coarser and inferior material on the back; the unequal contraction causes the two layers to separate.

Colour.—The colour varies with the temperature at which the clay is baked, and with the percentage of oxide of iron present. Colour may be imparted by giving the clay a wash of ochre paint before baking. This is inferior, soon wears, and should not be resorted to.

Two-colour Work.—Usually the blocks of terra-cotta are of one tint, but of late years manufacturers have perfected a process in which blocks each of two colours may be produced by carefully selecting the earth.

Specification.—This should include the following: the blocks to be sound and well burnt, with true faces, sharp arrises, with a vitreous surface of an even colour, no holes visible on face, and with no fire cracks or chips.

Fixing.—Terra-cotta blocks, when built in walls or anywhere to withstand pressure, have their voids filled up with ordinary Portland cement concrete; this usually causes discoloration due to the cement concrete working through. A filling composed of Portland cement and ground terra-cotta is more effective. The voids are left hollow in floors or any position where lightness is desirable.

Moulded work in terra-cotta, owing to the unequal shrinkage in drying, often becomes slightly twisted, which in buildings with a long run of cornice or string mouldings has a bad effect. This is often corrected by chiselling parts of the face; but it should not be allowed, as, if the vitrified surface skin be destroyed, the remainder will rapidly disintegrate under the influence of the weather.

The dimensions of the blocks are usually some multiple of brick dimensions, in order to bond with the brick backing.

Terra-cotta is most suitable for decorated panels, statuary, and work which would have to be carved if in stone, especially where these features have to be repeated several times.

STONEWARE

Stoneware is prepared from clays of the Lias formation, consisting of about 76 parts of silica and 24 parts of alumina, with a small percentage of iron, calcium, etc.

Good beds are found at Poole in Dorsetshire, Teignmouth in Devonshire, and other places.

The clay is prepared in a similar manner to the brick-earth, usually being mixed with a proportion of ground stoneware or sand to avoid excessive contraction in burning.

The articles are moulded by various processes, and burnt in a domed kiln ; the material is practically non-absorbent, but to ensure this the articles are glazed.

The glazing is performed by adding sodium chloride when in the kiln, which is volatilized by the heat, and in the form of a vapour is decomposed by the silicates of alumina, with which it combines to form a glass, the chlorine passing off from the kiln.

The sodium chloride in the form of a vapour penetrates into all the pores of the stoneware, completely covering the surface with a coating of glass.

This material is chiefly used for sanitary goods, including drain and sewer pipes, or where a damp-resisting material is required. These are burnt in kilns at a very high temperature, and when finished are thoroughly vitrified throughout their whole thickness.

EARTHENWARE

Earthenware.—Pipes and other goods usually made in stoneware are sometimes made from ordinary brick-earths such as are used in the manufacture of bricks and tiles and known as earthenware. These articles are weak and porous and especially liable to be damaged by frosts. Fire-clay ware made from fire-clays of the coal measures has similar characteristics ; is especially useful as a lining for flues subjected to great heat. Both earthenware and fire-clay goods should be glazed to prevent disintegration when exposed to the atmosphere or other destructive agencies.

CHAPTER VII

IRON AND STEEL

Ores.—Iron is extracted from ores, of which the following are the most abundant :—

Magnetic Iron Ore, or "Magnetite," a black oxide, when pure, yields 72·41 per cent. of metallic iron, the formula being Fe_3O_4. Swedish iron is obtained from this ore. It is very valuable, and is found in considerable uantities in many parts of Europe, America, and Asia.

Red Hæmatite is an oxide of iron containing 69·5 per cent. of iron, and is valuable as an iron-producing ore. Its formula is Fe_2O_3.

Brown Hæmatite is a hydrated ferric oxide. Its formula is $2Fe_2O_3 + 3H_2O$. It yields about 59·89 per cent. of metallic iron.

Spathic Iron Ores, Clay, or Cleveland Ironstone are names given to ferrous carbonate ores, $FeCO_3$, from which nearly two-thirds of the total weight of pig iron produced in Great Britain are smelted. The purer ores are known as spathic, whilst the amorphous argillaceous ores of the coal measures are known as clay ironstones, and when largely impregnated with carbonaceous or bituminous matter are called blackband ironstone. The spathic ore when pure yields about 48·27 per cent. of metallic iron.

Iron Pyrites is a bisulphide of iron (FeS_2), containing as much as 53·33 per cent. of sulphur, and is unfit for the extraction of iron.

The foregoing ores are more or less found in combination with impurities, such as manganese, magnesia, silica, alumina, lime, carbon, sulphur, and phosphorus, and seldom yield more than 33 to 66 per cent. of metallic iron.

CAST IRON

Production.—(*a*) The mechanical preparation of the iron ore, such as breaking up with the hammer, crushing and washing.

(*b*) The weathering of the ore; that is, leaving the ore in the open, exposed to the effects of the weather. This is not applicable to calcareous ores.

(*c*) Roasting or calcination to expel the water, CO_2, and other matters. This is generally accomplished by mixing the ore and fuel together, and setting fire to the mass.

(*d*) Smelting in the blast furnace to decarburize the ore, eliminating the carbon to 5 per cent. of the resultant product, which is pig iron. Fluxes in iron-smelting are used to combine with the earthy parts of the iron ore, and thus the pure iron is liberated. Limestone, clay, and sometimes sand is used as a flux. Charcoal and coke are principally used as fuel. Iron ores are frequently mixed together in the blast furnace, in order that the earthy matter of the different ores may act as fluxes without the addition of other materials.

Air is forced into the furnace by means of a blowing engine. In the older types of furnaces the air was not heated, and this was said to be a cold blast; but in the modern examples, the gases which previously were allowed to escape are utilized to heat the air, which is then forced into the furnace at heats of about 1,500° Fahr.; it is then termed the hot blast. Coke or charcoal are the only fuels employed in the cold blast, but the hot blast admits of the use of coal; and coke is also used. It generally is acknowledged that the hot blast may, with proper care, produce as good an iron as the cold blast; but the heat obtained in the former method is so great that it is possible to reduce refractory cinders and slags, therefore successful working requires skilful manipulation.

(*e*) Pig Iron is the production of the blast furnace. It consists of a combination of pure iron with carbon, both in chemical combination and mechanically mixed. Other substances, such as silicon, sulphur, phosphorus, and

manganese, are found in pigi ron. The pigs are often divided into six varieties. Nos. 1, 2, 3 are termed foundry pigs, and are used for the production of grey cast iron ; they are less fusible, but more fluid when molten than the whiter varieties, and have the property of expanding a little at the moment of solidification. Nos. 4, 5, 6 are described as forge pigs ; they are stronger and more brittle than the grey, and are used for conversion into wrought or pure iron.

(*f*) *Grey Cast Iron.*—This class of cast iron has the greatest ratio of uncombined or graphite carbon, and but a very small quantity of chemically combined carbon, viz., 2·9 to 3·7 per cent. crystallizes separately as graphite, and ·6 to 1·5 chemically combined.

(*g*) *Mottled Cast Iron* (No. 4 pig) has a larger proportion of the chemically combined carbon, and is stronger, whiter, and more lustrous than the grey varieties, has a granular and more or less mottled appearance on fracture, and is used only for the heaviest classes of foundry work. It is unsuitable for light or ornamental castings.

(*h*) *White Cast Iron* has nearly all its carbon in chemical combination. The quantity varies from 3 to 5 per cent.

(*i*) *Pig Irons* of various qualities are mixed together in the foundry cupola furnace to produce iron suitable for particular kinds of castings. All castings are usually specified to be of the second melting, but it has been proved by experiment to be better up to the twelfth remelting.

Characteristics.—Cast iron may be described as hard, brittle, fusible at 2,000° Fahr., a temperature easily obtainable in the blast furnace ; but it is not forgeable nor weldable.

Tests.—Cast-iron bars 1 inch square in section, and laid upon supports 1 foot apart, should resist a load of 1 ton placed in the centre of their length. The sound should be noted, and a close examination made to discover if any flaws or air bubbles exist. Chemical tests are of very little use, as indications of the quality must depend upon practical knowledge.

Dilute nitric acid applied to a clean fracture of grey iron will produce a black stain, and to a white iron a brown stain.

Uses.—Grey cast iron is mostly used in construction for columns, stanchions, short struts, and generally to resist compression, but is not suited to withstand tensile stresses.

Specifications for Constructional Cast Iron.—Except where chilled iron is specified, all castings shall be tough grey iron, free from injurious cold shuts or blow holes, true to pattern and of a workmanlike finish. Sample pieces 1 inch square cast from the same heat of metal in sand moulds shall be capable of sustaining on a clear span of 4 ft. 6 in. a central load of 500 lbs. when tested in the rough bar.

In some cases the sand forming the mould of the casting is specified to be baked in an oven, so that flaws that are caused through damp sand in the mould may be avoided.

WROUGHT IRON

The Production of Wrought Iron.—Wrought iron is nearly the pure metal, containing usually not more than 0·15 per cent. of carbon.

(*a*) Refining consists in keeping the pig iron in a state of fusion on an open hearth with coke or charcoal while a blast of atmospheric air from several inclined twyers is at the same time directed upon the surface of the molten metal. It is an operation having for its object the decarburization and purification of pig iron, which it only partially accomplishes, and produces fine or plate metal, and before conversion into malleable iron must be puddled. This process is now rarely employed.

(*b*) Puddling, or Pig Boiling in the puddler's reverberatory furnace, to obtain pure iron by eliminating the carbon and other foreign substances, sufficiently removes the carbon and the impurities, producing puddled balls. Dry puddling is the name given to the process when refined iron is employed, and pig boiling when raw or pig iron is used direct. The extent to which most of the foreign sub-

stances are removed from the pig iron by puddling may be inferred from the comparison of the composition of a puddled bar with that of the good No. 3 grey cold blast Staffordshire pig, from which it was obtained :—

	Pig Iron.	Puddle Bar.
Carbon	2·28 ...	0·30
Silicon	2·72 ...	0·12
Phosphorus	0·65 ...	0·14
Sulphur	0·30 ...	0·13
Iron	94·05 ...	99·31

(*c*) Shingling, or Bloomitig, or crushing with the steam hammer, to force out the cinder, and consolidate and weld the particles of iron together, and rolling to produce puddle bar.

(*d*) No. 1, Rough or Puddled Bar Iron, is coarse and brittle, of a low tensile strength, usually employed in the production of better qualities of iron.

(*e*) No. 2, or Merchant Bar Iron, also known as common iron, is the lowest quality used by the smiths ; it is hard and brittle, and only fit for rough work, is produced by piling up puddled bars, raising them to a welding heat, and passing them through rollers.

(*f*) No. 3, B or Best Iron, is produced from No. 2 by the process of piling, reheating, and rolling. This quality is used for ordinary good work.

(*g*) BB Iron is produced from No. 3 by a similar process of piling, reheating, and rolling ; this quality, and also BBB Iron, is used for special purposes.

Rolling.—The particles of wrought iron after reheating or exposure to a high temperature assume or revert to a crystalline state of a cubical form. Hot rolling converts these crystals into fibres, nearly doubles the tensile strength, but reduces the ductility of the metal. After the fifth reheating, the iron loses instead of gains in tensile strength. Cold rolling puts a polish on the outside surface of the metal, making it harder than the interior ; in all other respects it has a similar effect upon the iron as mentioned for hot rolling.

Characteristics.—Wrought or Pure Iron may be described as soft, malleable, ductile, weldable at white heat (at 1,500°

to 1,600° Fahr., it softens and can be welded), easily forgeable, very tenacious, but not fusible except at high temperatures, viz., 3,000° Fahr., and is not temperable.

Tests of Resilience.—The quality is usually decided by noting its strength and ductility by taking specimens about 10 inches in length and subjecting them to tension in a testing machine. The standard required for constructional purposes is that it should elongate 20 per cent. of its length under a slowly applied tensile-breaking stress; thus, a specimen 1 inch in sectional area and 10 inches in length should measure 12 inches in length at time of rupture, and fail under a gradually increasing stress of not less than 22 tons per square inch. The value of the resilience will thus be equal to 22 inch tons, being the given average resistance multiplied by the increase of length.

Sometimes the iron is specified to be bent through a given angle without cracking, sometimes hot and sometimes cold; the better the iron the more it can be bent, and the test is more severe when the operation has to be performed cold. Thus, rivet iron is specified to be bent double when cold without cracking.

Dilute nitric acid applied to a clean fracture will leave a greenish stain.

Uses.—Wrought iron is of a fibrous nature, and is therefore very suitable to resist tensile stresses, and has been much used for roofs, girders, and long stanchions. For the latter purpose, the reliable nature of the material is a great recommendation, but it is now displaced by mild steel.

STEEL

Definition.—All combinations of iron and carbon which are malleable and permit of being hardened and tempered, and capable of being cast into a malleable ingot, may be considered as steel.

Classification.—Steel may be divided under two heads: mild or soft, and hard. Mild steels include shear, double shear, and those made by the Bessemer and Siemens

processes. These may be employed for all constructional purposes, but the expense of shear and double shear generally precludes their use, so the Bessemer and Siemens are now almost universally employed for work where great strength, tenacity and ductility are required without any very great hardness. Hard steel includes the crucible cast, especially useful for cutting tools, and the Whitworth compressed ingot, which, being compressed in the liquid state with a hydraulic force varying from 10 to 20 tons per square inch, reduces the number of small gas cells to which ordinary cast steel is usually subject.

Production.—There are three distinct systems adopted in the manufacture of mild steel.

First, the cementation process, viz., that of refining cast iron into an almost pure wrought iron, and then afterwards combining the pure wrought iron with a definite amount of carbon, which it absorbs from the charcoal, and thus gives to iron the nature of steel.

Bars of wrought iron embedded in charcoal are exposed to white heat (2,142° Fahr.) in a cementation furnace ; the time occupied by the process depends upon the quality of the steel required—it generally requires from one week to a fortnight. The converted bars are classed as blistered steel. From this quality, by a repetition of the three processes piling, reheating, and welding, are produced single shear steel and double shear steel.

Spring Steel is blistered steel heated to an orange-red colour and hammered or rolled.

Single Shear, or Tilted Steel, is produced from bars of blistered steel, piled into bundles, placed at a welding heat under the tilt hammer or the steam hammer, which removes the blisters, closes the seams, and forms one bar of single shear steel. The process is again performed upon single shear to produce double shear steel.

Crucible Cast Steel is made by melting blister steel in covered fire-clay vessels, and running the metal into iron moulds. In another method, Swedish wrought-iron bars of good iron are melted into the vessel with charcoal.

Secondly, the process introduced by Sir Henry Bessemer, in which a volume of dense air is forced through

the crude cast iron in the molten state. During its passage the oxygen combines chemically with the carbon, and carbon dioxide passes off, thus leaving the iron comparatively pure. In order to make it into steel there is added to the purified metal a measured portion of pure cast iron, commonly called "Spiegeleisen," or "looking-glass iron." An analysis of one sample of this contained 82·86 iron, 10·71 manganese, 1·0 silicon, 4·32 carbon—total 98·89 parts. It is introduced in the melted state in the proportion of 1 part to 30 parts of the pig iron employed. The vessel employed for the fusion of the molten ores is called a converter, and is often made capable of containing 6 to 10 tons of material. The operation takes about 20 minutes. This steel is produced at a much less cost than by the cementation process.

Thirdly, the Siemens, or open hearth process, consists in mixing and heating an iron ore, rich in oxide, with the mass of crude iron, and sometimes steel scrap. In this, the oxygen of the ore and the lining performs the same chemical office as that from the air in the Bessemer process, and, uniting with the carbon in the crude iron, passes off as carbon dioxide. The lining, siliceous or basic, is made good each time. The furnace used is of the kind known as a regenerator. Symmetrically, about the centre are arranged a number of flues, so that at any given moment of working those flues serve on one side to lead away the products of combustion to the chimney shaft, the escaping gases on their journey giving up considerable heat to the brick flues; on the other side the flues form the passage for the fuel, which consists of producer gas and heated air, to the hearth or combustion chamber, the heated flues being cooled by giving up their heat to the charge of producer gas and heated air, making them very hot as they pass over the combustion zone. Every few minutes, by the operation of a valve, the function of the flues on each side is exchanged.

By this device, which is under complete control, a very great amount of heat is utilized which would otherwise be wasted. By means of this system manufacturers are enabled to produce steel of any degree of softness, suitable for constructional work generally. The best steel for high-

class cutlery and tools is produced by the cementation process. The Bessemer turns well, but the time taken in the Siemens process, which is about six hours for each charge, allows tests to be made, and a more uniform quality is ensured. The Bessemer steel is slightly cheaper than that produced by the Siemens process.

Processes.—Both the Bessemer and Siemens processes are divisible into two classes: first, that conducted in acid (siliceous) lined vessels or furnaces in which only the purer kinds of pig iron can be operated upon; and the second, in similar vessels or furnaces, but with basic (dolomitic) linings, which are capable of using the more impure phosphoric pig irons. In the Bessemer process the acid lining serves for about 220 charges; the basic lining for a lesser number.

ANALYSES OF VARIOUS STEELS FROM STEEL AND IRON.—W. H. GREENWOOD.

	Soft Neuberg-Bessemer Steel.	Soft Siemens-Martin Steel.	Siemens Steel Plates.	Bessemer Steel Rails.	Siemens Steel Rails.	Crucible Steel for Forgings.	Bessemer Steel Rails.	Hard Bessemer Steel.	Hard Tool Steel.
Carbon . .	·126	·167	·21	·352	·370	·36	·313	·687	1·144
Silicon . .	·135	·023	·047	·053	·040	·02	·078	·046	·166
Sulphur. .	·014	·013	·052	·055	·042	·02	·076	·008	...
Phosphorus	·060	·062	·035	·061	·033	·03	·071	·036	...
Manganese.	·158	·044	·36	·384	·342	·30	·515	·404	·104
Copper . .	·112	·076	...	trace	·008	trace	trace	·119	...

Characteristics.—Steel may be described as highly elastic, malleable, ductile, forgeable, weldable, capable of receiving different degrees of hardness by tempering, and fusible at a lower temperature than wrought iron (at 2,400° Fahr.); hardened steel is noted for its property of retaining magnetism. Generally speaking, the smaller the amount of carbon steel contains, the nearer will its properties resemble those of wrought iron; the greater the quantity it possesses tends to make its characteristics similar to cast iron. In the softer or milder steels the structure approaches more closely to a fibrous condition. In the harder varieties the structure shows it to be fine, shining, and uniformly granular, the grains being in lines perpendicular to the sides of the ingot.

Tests.—Steel should be tested for strength and ductility, which is known as the resilience test; thus a mild steel suitable for construction should possess a tenacity (maximum load divided by initial cross sectional area of specimen) under a slowly applied tensile stress of 30 to 32 tons per square inch, and the metal should elongate 20 per cent. of its length before fracture takes place. Thus, a bar of 1 inch sectional area and 10 inches long will stretch to 12 inches before rupture, a gradually increasing force up to 30 tons being required to cause the same. The sound should also be noted, and sometimes a forged test is required, viz., to bend hot or cold, with or across the grain, without fracture, through angles varying 20 to 90 degrees. Dilute nitric acid applied to a clean fracture of steel will leave a dark grey stain, owing to the separation of carbon.

Patterns.—A smaller allowance for shrinkage is required in patterns for steel castings than for cast iron.

Uses.—Mild steel is suitable for constructional purposes generally, members for bridges, girders, roofs, etc., and has practically displaced the use of wrought iron for constructional purposes. Hard cast steel is used for cutting tools.

Hardening and Tempering.—These are distinctive properties of steel. Mild steel, containing from 0·2 to 0·5 per

cent. of carbon, will weld, but does not temper, and will elongate from 25 to 30 per cent. of its length before fracture under a maximum tensile stress of 28 to 33 tons per square inch. When the carbon reaches 1·5 to 2 per cent., it is at the expense of tenacity and weldability of the material; but these harder varieties are better suited for the process of tempering. Hardening lowers the specific gravity, but increases the tenacity.

Smithing.—The smithing of steel is more difficult than that of wrought iron, and it is more liable to injury from over-heating, therefore necessitating greater care.

Punching.—Steel plates sustain greater injury when punched than wrought iron. Occasionally in punching, a fracture is produced, extending to the edge of the plate. Experiments made show that the loss of strength of plates with drilled holes is less than that of similar plates punched; if punched, steel plates should be annealed, when the greater part of their strength is restored; but to resist a shearing stress, drilled holes are 4 per cent. weaker than those punched.

The usual practice with steel plates is to punch those with a thickness of less than $\frac{1}{2}$ inch, and not to anneal them afterwards. Plates $\frac{1}{2}$ to $\frac{3}{4}$ inch in thickness are punched, and rymered or annealed afterwards; plates more than $\frac{3}{4}$ inch should be drilled. The tendency of modern practice is now to drill all plates.

Fracture.—Cast iron may be described as crystalline, wrought iron as fibrous, and steel as of a granular nature. If a breaking stress be applied slowly, the fractures upon the representative specimens of each class will be: Cast iron, crystalline; wrought iron, fibrous; and steel, silky fibrous; but if a breaking stress is suddenly applied, it will tend to cause all the fractured specimens to be crystalline; thus, cast iron will be very crystalline, wrought iron less crystalline, and sometimes fibrous threads will be interwoven, and steel invariably granular.

Sound.—Cast iron, when struck, will give comparatively a hollow sound ; wrought iron, a note of a low pitch ; but steel will give a distinctly clear treble ring.

Phosphorus is largely taken up by iron during the process of smelting. Its effect upon cast iron is to harden it, to render it more fusible, but to reduce its tenacity. Wrought iron, with $\frac{1}{10}$ per cent. of phosphorus, is better for welding. Half per cent. makes the metal cold short ; that is, brittle at low temperature, and cracking when bent. Antimony and tin will have a similar effect. Steel is injured by the presence of phosphorus even in the smallest quantities.

Sulphur in cast iron tends to produce the mottled and white varieties ; in wrought iron $\frac{3}{10}$ to $\frac{4}{10}$ per cent. produces red or hot shortness, that is, causes the metal to be brittle at high temperatures. In steel, $\frac{2}{10}$ per cent. renders it unfit for forging, but makes it more fluid and better for casting. One-tenth per cent. produces red shortness.

Manganese is nearly always present in cast iron. It tends to produce the white variety, in which a large proportion is generally found. In wrought iron and steel it counteracts red shortness, most probably by encouraging the departure of sulphur and silicon, and is essential in the manufacture of Bessemer and Siemens steel.

Copper in cast iron to the extent of $\frac{2}{10}$ per cent. does no harm. In wrought iron $\frac{1}{2}$ per cent. makes it red short. In steel $\frac{1}{2}$ per cent. makes it red short. Two per cent. causes it to be brittle.

Case-Hardening.—Wrought iron may have its outer crust partially converted into steel by either of the following processes, when it is said to be case-hardened : The surface of the iron is made bright ; it is then placed in a very clear fire until red-hot, then rubbed with powdered yellow prussiate of potash, and then reheated until the iron assumes a cherry-red heat, when it is cooled suddenly by being immersed in water. It is more effectually case-

hardened by placing the bright iron in a close iron box filled with bone dust and cuttings of horn and leather substances, which part with their carbon (sometimes common salt is added), and heating for 24 hours in a fire. This method is employed in small parts, such as pins, where the wearing property of steel is desired to be combined with the ductility of wrought iron.

Malleable Iron.—Small articles of cast iron are sometimes made partially or wholly malleable by surrounding the casting with an oxidizing compound, such as oxide of iron or powdered red hæmatite, and keeping it at a high temperature for a time ranging from 2 to 40 hours, varying with the size of the casting, to eliminate the carbon and convert it into a material resembling wrought iron. Decorative parts of ironwork to withstand blows are often treated in this manner. Castings of $\frac{1}{2}$ inch in thickness are rendered malleable throughout; thicker castings have only the skin rendered malleable.

Preservation of Iron.—Cast iron should be painted soon after it leaves the mould to preserve intact the hard skin before it has time to rust. Lead paints are often used, but as galvanic action is set up, oxide of iron paints should be preferred. Cast-iron pipes are effectually treated by Dr. Angus Smith's process, which consists in cleaning the castings and heating them to 700° Fahr., and dipping them in a mixture heated to 300°, consisting of coal tar, pitch, 5 per cent. of linseed oil, and a little resin.

Wrought-iron sheets are effectively protected by being galvanized, which consists in cleaning the iron with sulphuric acid, scouring with sand, and washing until clean with water, and then covering the iron with a thin coating of zinc. Wrought iron and steel are preserved by the process of superheated steam (Barff's process) and superheated air (Bower's process), which latter is much cheaper for cast iron. These processes cover the iron with a coating of black magnetic oxide, Fe_3O_4 with more or less Fe_2O_3 and some Fe_4O_5, which effectually resists oxidation from damp earth, salt water, and other causes.

Steel rods covered w th a coating of Portland cement

	Specific Gravity.	Weight per ft. cube in lbs.	Tenacity in tons per square inch. (Maximum Stress).			Safe working loads in tons per square inch for dead load.			Modulus of Elasticity in tension. Lbs. per square inch. (E)	Yield Stress.	Chemical Analysis.
			Compression.	Tension.	Shearing.	Compression.	Tension.	Shearing.			
Cast Iron	7·2	450	40 to 45	7 to 8	12	8 to 9	$1\frac{1}{2}$ to $1\frac{3}{4}$	2	18,000,000	$\frac{1}{8}$ to $\frac{1}{5}$ of ultimate strength	Contains 3 to 5 per cent. of carbon
Wrght. Iron	7·6	480	16 to 17	20 plates 25 bars	22	4	5	4	25,000,000 to 29,000,000	$\frac{1}{2}$ of ultimate strength	Not more than ·15 chemically combined carbon
Steel, Mild	7·8	490	28 to 33, 32 usually employed	28 to 33, 32 usually employed	24	8	8	5	30,000,000	0·45 to ·8 of ultimate strength	·15 to 2 per cent. of chemically combined carbon

grouting, and surrounded with watertight concrete, is found effectually to preserve steel, and is practised on a large scale in ferro-concrete construction.

Oxidation.—The comparative oxidation or rusting of cast iron, wrought iron, and steel in moist air (ordinary rust of iron $2Fe_2O_3$, $3H_2O$), are respectively as 100, 129, 133. Cast iron is, therefore, the best of these materials to resist oxidation.

BRITISH STANDARD SPECIFICATION FOR STRUCTURAL STEEL FOR BRIDGES, ETC., AND GENERAL BUILDING CONSTRUCTION.

[REVISED MAY, 1930.] No. 15—1930

NOTE.—*The Institution desires to call attention to the fact that this Specification is intended to include the technical provisions necessary for the supply of the material herein referred to, but does not purport to include all the necessary provisions of a contract.*

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The figures in British measures are to be regarded as the Standard. Approximate metric equivalents are given for the convenience of users in countries in which the metric system has been generally adopted.

FOREWORD

The British Standard Specification for Structural Steel for Bridges and General Building Construction (B.S.S. No. 15) was first issued in 1906 and a revised edition appeared in 1912.

The following are the principal alterations which have been made in the present edition of the Specification :—

(i.) All reference to the Basic Bessemer process is now omitted.

(ii.) Drawn steel wire from 0·5 to 0·125 inches in diameter which has subsequently been suitably heat treated to enable it to conform with the requirements of this Specification is now included. Hard drawn Steel wire for concrete reinforcement, not so treated, is dealt with under Specification No. 165.

(iii.) A tensile test in addition to a bend test is required for bars for concrete reinforcement.

(iv.) Temper bend tests are not now specified.

(v.) The radius of the bend in the cold bend test has been made smaller for bars of 1 inch diameter and under.

(vi.) Tolerances on the specified depth of beams and channels have been inserted.

NOTE.—This Specification shall apply to Drawn Steel Wire from 0·5 to 0·125 inches (12·70 to 3·18 mm.) in diameter which has subsequently been suitably heat treated and when so applied the words "bar" or "bars," where they occur, shall be deemed to include such wire and shall be read as "wire" or "wires," "coil of wire" or "coils of wire" as the context may require.

1. *Process of Manufacture.*—**A** steel shall be made by the open hearth process (acid or basic), unless either process is required or specified, and shall not show on analysis more than 0·06 per cent. of sulphur or of phosphorus.

B steel may be made either by the open hearth process (acid or basic), or by the acid Bessemer process, and shall not show on analysis more than 0·08 per cent. of phosphorus, and not more than 0·06 per cent. of sulphur.

NOTE.—**B** steel is not intended for Bridges, Plates $\frac{1}{4}$ inch in thickness and over, Rivet Bars, or for Heat Treated Wire.

2. *Quality of Finished Steel.*—All finished steel as sent from the mills shall, subject to the provisions of Clause 17, be well and cleanly rolled to the dimensions, sections and weights specified or required. It shall be sound and free from cracks, surface flaws, laminations, rough, jagged and imperfect edges and all other defects, shall be finished in a workmanlike manner and shall in all respects comply with the tests and requirements, herein mentioned, applicable to the description of material (*e.g.*, plates, sections, bars, rivets, etc.) required or specified.

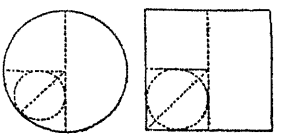
3. *Tensile Test Pieces.*—The tensile strength and elongation of all steel shall be determined from standard test pieces cut lengthwise and crosswise from plates, and lengthwise from sections and bars.

The test pieces shall not be annealed, or otherwise subjected to heat treatment, unless the material from which

they are cut is similarly treated, in which case the test pieces shall be similarly and simultaneously treated with the material before testing.

Any straightening of test pieces which may be required shall be done cold.

The rolled surface of the steel wherever practicable shall be retained on two opposite sides of the test piece, but in the case of bars having diameters or sides not exceeding 3 inches (76·20 mm.) the bars may be reduced by machining. For bars having diameters or sides above 3 inches the test piece may, at the option of the Maker, be taken from the position shown in the sketches.



4. *Selection of Tensile Test Pieces.*—Tensile test pieces shall be selected by the Purchaser or by the Engineer* or Inspector† either—

(*a*) From shearings or cuttings of the plates, sections, and bars, or

(*b*) If he so desire, from the plates, sections, and bars, after they have been cut to the sizes required or specified.

In the latter case (*b*), if the test is satisfactory, the Purchaser shall pay the Maker the value of the plate, section, or bar from which the test piece has been cut, or accept delivery of the same as though such test piece had not been cut therefrom.

In neither case (*a* or *b*) shall the test pieces be detached from the plates, sections, or bars, except in the presence or with the approval of the Purchaser or of the Engineer or Inspector.

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* The word " Engineer " shall mean the Engineer or Architect supervising or acting as Engineer or Architect for the Purchaser.

† The word " Inspector " shall include any person acting under the direction of such Engineer or Architect.

5. *Tensile Tests.*—The tensile breaking strength of all steel determined from the standard test pieces hereinafter referred to, shall be as follows :—

(*a*) *Plates, Sections* (e.g., *Angles, Tees, Joists, Channels, etc.*), *and Flat Bars.*—The tensile breaking strength of all plates, sections (such as angles, tees, joists, channels, etc.) and flat bars, shall be between the limits of 28 and 33 tons (62,720 and 73,920 lbs.) per square inch (44·10 and 51·97 kg. per mm².) of section. The elongation measured on the Standard Test Piece **A** (see Appendix, page 168) shall be not less than 20 per cent. for steel of 0·375 inch (9·53 mm.) in thickness and upwards, and not less than 16 per cent. for steel below 0·375 inch in thickness. In the case of sections the thickness of which is not uniform throughout the profile, these limits shall be applied according to the actual maximum thickness of the piece selected for testing.

(*b*) *Round and Square Bars.*—The tensile breaking strength of round and square bars (other than rivet bars) shall be between the limits of 28 and 33 tons per square inch of section, with an elongation of not less than 20 per cent. measured on the Standard Test Piece **B** (see Appendix, page 169), or not less than 24 per cent. measured on the Standard Test Piece **F** (see Appendix, page 170). The bars may be tested the full size as rolled.

(*c*) *Rivet Bars.*—The tensile breaking strength of rivet bars shall be between the limits of 25 and 30 tons (56,000 and 67,200 lbs.) per square inch (39·37 and 47·25 kg. per mm².) of section, with an elongation of not less than 25 per cent. measured on the Standard Test Piece **B** (see Appendix, page 169) or not less than 30 per cent. measured on Standard Test Piece **F** (see Appendix, page 170). The bars may be tested the full size as rolled.

Generally.—Provided that, except for bars for concrete reinforcement, bend tests only shall be required for steel under $\frac{1}{4}$ inch (6·35 mm.) in thickness or diameter.

6. *Number of Tensile Tests.*—(A) *For Tensile Tests Clause* 5, *Sub-Section* (*a*).—One tensile test shall be made from the finished steel from each cast for any quantity up to 25 tons (56,000 lbs. = 25,400 kg.) of plates, each type of section, and flat bars rolled from that cast, a separate

test being made for each class (*e.g.*, plates, types of sections, and flat bars). A second tensile test shall be made of the material in any class when the quantity in that class exceeds 25 tons.

Where plates, types of sections, or flat bars of more than one thickness are rolled from the same cast, one additional tensile test shall be made from the material in each class for each variation in thickness of 0·2 inch (5·08 mm.) above or below the thickness of the test piece first selected in such class.

(B) *For Tensile Tests, Clause* 5 *Sub-Section* (*b*).—One tensile test shall be made from the finished steel from each cast for any quantity up to 25 tons, and a second tensile test shall be made where the quantity exceeds 25 tons. When more than one diameter or thickness of bar is required or specified one additional tensile test shall be made from each diameter or thickness of bar ordered, if desired by the Purchaser or by the Engineer or Inspector.

NOTE.—For heat treated wire the words " 10 coils of wire " shall be substituted for " 25 tons " in this sub-section.

(C) *For Tensile Tests Clause* 5, *Sub-Section* (*c*).—One tensile test shall be made from the finished steel from each cast for any quantity up to 10 tons (22,400 lbs. = 10,160 kg.) and a second tensile test shall be made for each further 10 tons or part thereof, from that cast.

NOTE.—For heat treated wire the words " 10 coils of wire " shall be substituted for " 10 tons " in this sub-section.

7. *Cold Bend Test Pieces.*—Bend tests of all steel (other than rivet bars) shall be made from test pieces prepared as follows: Bend test pieces shall be sheared or cut lengthwise and crosswise from plates and lengthwise from sections and round and square bars and, when the section permits, shall be not less than 1½ inches (38·10 mm.) wide. In cases where the section is less than 1½ inches wide, or if the Maker so desires, round, square and flat bars shall be bent in the full section of the bar as rolled.

In all bend tests the rough edge or arris caused by shearing may be removed by filing or grinding, and samples 1 inch (25·40 mm.) in thickness and above may have the

edges machined, but the test pieces shall receive no other preparation.

The test pieces shall not be annealed, or otherwise subjected to heat treatment, unless the material from which they are cut is similarly treated, in which case the test pieces shall be similarly and simultaneously treated with the material before testing.

8. *Selection of Cold Bent Test Pieces.*—Bend test pieces shall be selected by the Purchaser or by the Engineer or Inspector either—

(*a*) From shearings or cuttings of the plates, sections, and bars, or

(*b*) If he so desires, from the plates, sections, and bars, after they have been cut to the sizes required or specified.

In the latter case (*b*), if the test is satisfactory, the Purchaser shall pay the Maker the value of the plate, section, or bar from which the test piece has been cut, or accept delivery of the same as though such test piece had not been cut therefrom.

9. *Cold Bend Tests.*—For cold bend tests, except in the case of round bars 1 inch (25·40 mm.) in diameter and under, the test piece shall withstand, without fracture, being doubled over either by pressure or by blows from a hammer until the internal radius is not greater than $1\frac{1}{2}$ times the thickness of the test piece, and the sides are parallel.

In the case of round bars, 1 inch in diameter and under, the internal radius of the bend shall be not greater than the diameter of the bar.

For sections having flanges less than 2 inches (50·80 mm.) wide these bend tests may be made from the flattened section.

10. *Number of Cold Bend Tests.*—A cold bend test shall be made from each plate, section, or bar (other than rivet bars) as rolled.

For rivet bars bend tests are not required.

11. *Number and Kind of Tests of Manufactured Rivets.*—Manufactured rivets selected from the bulk, in such number as may be specified, or as may be approved by the Purchaser or by the Engineer, shall withstand the following tests :—

(*a*) The rivet shanks shall be bent cold, and hammered until the two parts of the shank touch in the manner shown in Fig. 1, without fracture on the outside of the bend.

(*b*) The rivet heads shall be flattened, while hot, in the manner shown in Fig. 2, without cracking at the edges. The head shall be flattened until its diameter is $2\frac{1}{2}$ times the diameter of the shank.



Fig. 1.

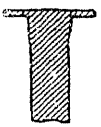


Fig. 2.

12. *Tests by Chemical Analysis.*—The Maker shall supply an analysis of each cast of steel when required so to do by the Purchaser or by the Engineer, but samples may also be taken by the Purchaser or by the Engineer or Inspector, and at the expense of the Purchaser may be subjected to complete analysis by a metallurgist appointed by him.

13. *Maker's Tests at his Works.*—All the test pieces after they have been marked for testing shall (except as provided in Clause 14) be prepared by the Maker and tested at his works and at his cost, and, if the Purchaser or Engineer so desire, in the presence of the Purchaser or of the Engineer or Inspector. If the Maker fails to prepare properly the test pieces for testing or to test the steel properly at his works in the manner herein provided, the Purchaser or the Engineer may, at the Maker's cost, have the test pieces prepared for testing and the steel tested elsewhere.

14. *Purchaser's Tests Elsewhere.*—Four days' notice shall be given by the Maker to the Purchaser and to the

Engineer of the date when he will be ready for the Purchaser or the Engineer or Inspector, to select the test pieces. If within 7 days after the receipt of such notice the Purchaser or the Engineer shall give notice in writing to the Maker that he desires the test pieces to be prepared and tested at a place to be named in the notice within the United Kingdom, the tests shall then be carried out at such place and at the Purchaser's cost (and if the Maker so desire in the presence of the Maker or of the person deputed by him to witness the tests). In the event of such notice being given by the Purchaser or by the Engineer, the Maker shall be allowed at least 7 days from the receipt of such notice before the selection of the test pieces. The tests must be completed within 14 days from the selection of the test pieces, failing which the tests shall be carried out at the Maker's works.

15. *Additional Tests.*—Should a tensile test piece break outside the middle-half of its gauge length, the test may, at the Maker's option, be discarded, and another test be made of the same plate, section or bar. In all other cases should any one of the test pieces or rivets first selected not fulfil the tests applicable to the description of material to be tested, two additional test pieces or rivets in respect of each failure may be taken from the material represented by that test, and should either of them fail to fulfil such tests, all the material so represented may be rejected. The additional tests shall be carried out in the same manner in all respects as the tests hereinbefore required to be made in the first instance, but at the cost of the Maker.

16. *Inspection.*—The Purchaser, the Engineer and the Inspector shall at all reasonable times have free access to the Maker's works, and to all places under his control where steel is being manufactured, and shall be at liberty to inspect the manufacture of the steel at all stages.

17. *Margin Over and Under Dimensions and Weights.*—(*a*) *Specified Lengths.*—When steel in bars or sections is specified to be cut to certain lengths it shall be cut within a margin of 1 inch (25·40 mm.) under or 1 inch over the

specified length, but when minimum lengths are specified the margin shall be within 2 inches (50·80 mm.) over.

(*b*) "*Exact*" *Lengths.*—When lengths are specified to be "exact," the steel in bars or sections shall be cold sawn or machined within a margin of $\frac{1}{8}$ inch (3·18 mm.) over and $\frac{1}{8}$ inch under the length specified.

(*c*) *Weights.*—When a minimum weight is specified the rolling margin on plates, sections and bars shall be 5 per cent. over, and when a maximum weight is specified the rolling margin shall be 5 per cent. under the specified weight.

When the specified weight is not stated to be either a minimum or maximum, the rolling margin shall be between $2\frac{1}{2}$ per cent. over and $2\frac{1}{2}$ per cent. under the specified weight.

The margin shall be ascertained separately for plates, each section (*e.g.*, angles, tees, beams, channels, etc.) and bars.

NOTE.—In the case of heat treated wire the tolerance shall be determined on the diameter and not on the weight. When a minimum diameter of wire is specified the tolerance shall be 2 per cent. over and where the maximum diameter is specified the tolerance shall be 2 per cent. under the specified diameter. When the specified diameter is not stated to be a minimum or a maximum the tolerance shall be 1 per cent. over and 1 per cent. under the specified diameter. Heat treated wire may be sheared to length.

(*d*) *Cross Sectional Dimensions of Beams and Channels.*—The permissible upwards and downwards variation in the specified depth of beams and channels shall not exceed the following :—

Specified Depth of Beam or Channel.	Variation.	
	Upward.	Downward.
Up to and including 12 in. (305 mm.)	$\frac{1}{8}$ in. (3·18 mm.)	$\frac{1}{32}$ in. (0·79 mm.)
Over 12 in. and up to and including 16 in. (406 mm.)	$\frac{5}{32}$ in. (3·97 mm.)	$\frac{1}{16}$ in. (1·59 mm.)
Over 16 in. and up to and including 24 in. (610 mm.)	$\frac{3}{16}$ in. (4·76 mm.)	$\frac{1}{16}$ in. (1·59 mm.)

18. *Calculation of Weight.*—The weight of plates shall be calculated on the basis that steel weighs 40·8 lbs. per square foot per inch of thickness (78·43 kg. per m^2., 1 cm. thick) and the weight of sections and bars on the basis that steel weighs 3·4 lbs. per square inch of sectional area per foot run (0·7843 kg. per cm^2. per metre run).

19. *Identification of Cast.*—The Maker shall mark the ingots, billets, slabs, plates, sections, bars, etc., in such a way as to enable all finished steel to be traced to the original cast. Every facility for tracing the steel to the original cast shall be given to the Purchaser and to the Engineer and Inspector.

20. *Branding or Marking.*—Every piece of steel shall be legibly marked with the Maker's name or trade mark, and with cast numbers or identification marks by which the steel can be traced to the cast from which it was made, except that in the case of such bars and small pieces as are securely bundled, a metal tag attached to each bundle and marked as above will be sufficient.

Before the test pieces are selected, the Maker shall furnish the Purchaser with copies of the mill sheets, giving complete lists of all plates, sections or bars in each cast, with sizes and weights, and the numbers or marks by which each plate, section or bar can be identified.

21. *Maker's Certificate.*—(*a*) *When no Inspection has taken place.*—In the case of any steel which has not been inspected at the Maker's works, the Maker or Merchant, as the case may be, shall supply the Purchaser and the Engineer with a certificate stating the process of manufacture and a test sheet signed by the Maker giving the results of each of the mechanical tests applicable to the description of material purchased, and if and when required of the chemical analysis also. Each test sheet shall indicate the numbers or identification marks of the casts to which it applies, corresponding with the numbers to be found on the plates, sections, bars, etc.

(*b*) *When Steel is taken from Stock.*—Where any steel is taken from a Merchant's stock, the Purchaser or the

Engineer may either (i) have the steel tested at such place as is in Clause 14 provided or (ii) the Merchant shall satisfy the Purchaser or the Engineer by means of numbers or identification marks on the steel, combined with a Maker's certificate, that such steel has been tested, and complies with the whole of the tests and requirements of this specification applicable to the description of material required or specified.

22. *Non-compliance with Tests and Requirements.*—Should any steel not comply with the whole of the foregoing tests and requirements applicable to the description of material required or specified, all the steel in the cast from which the tests have been taken may, subject to the option of making additional tests as provided in Clause 15, be rejected.

23. *Delivery.*—No steel shall be despatched from the Maker's works until it has been tested and complies with, or has been certified (in the cases mentioned in Clause 21) to comply with, the whole of the tests and requirements of this specification applicable to the description of material required or specified.

24. *Rejection after Delivery.*—The foregoing tests shall, except as provided for in Clause 14 and in Clause 21, Sub-Section (*b*), be made at the Maker's works prior to despatch, but in the event of any of the steel being found not to be in accordance with this specification in the course of being worked, such steel may be rejected, notwithstanding any previous acceptance, provided that the steel has not been improperly treated in working.

25. *Arbitration.*—In case any dispute shall arise between the parties to any contract in which this specification is in whole or part incorporated as to whether any process of manufacture required or specified, or any test or requirement of this specification has or has not been carried out or complied with, or as to whether any steel is or is not of the quality or free from the defects mentioned in Clause 2, or as to whether any steel proves unsatisfactory in the course

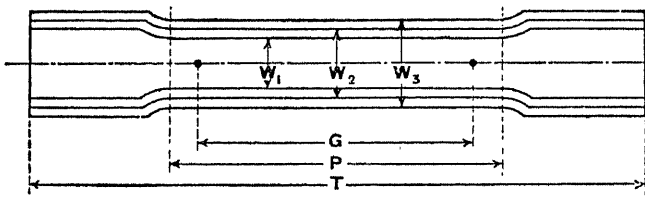
of being worked as provided in Clause 24, then such dispute shall be referred to the arbitration of a person to be agreed upon between the parties or failing agreement to be appointed, at the request of either party to the said dispute, by the Chairman for the time being of the British Engineering Standards Association, provided always that if such dispute is within the terms of any other agreement to refer or submit to arbitration this clause shall be of no effect.

NOTE.—Hard drawn steel Wire for concrete re-inforcement is dealt with in British Standard Specification No. 165.

APPENDIX I

FORMS OF BRITISH STANDARD TENSILE TEST PIECES

TEST PIECE **A.**

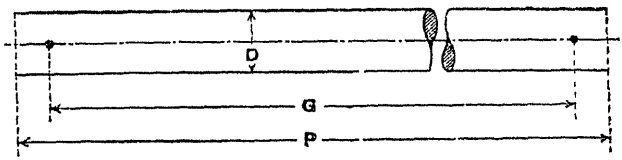


Gauge Length **G** = 8 inches (203·20 mm.).
Parallel Length **P** to be not less than 9 inches (228·60 mm.).
Total Length **T** = About 18 inches (457·20 mm.).

Thickness of Test Piece.	Maximum Width allowed.
Over $\frac{7}{8}$ in. (22·23 mm.)	W_1 = 1½ ins. (38·10 mm)
$\frac{3}{8}$ in. to $\frac{7}{8}$ in. (9·53 to 22·23 mm.)	W_2 = 2 ins. (50·80 mm.)
Under $\frac{3}{8}$ in. (9·53 mm.)	W_3 = 2½ ins. (63·50 mm.)

The widths of the test pieces for plates were selected to comply with the two following conditions. (1) As the great bulk of plates to be tested are from $\frac{3}{8}$ to $\frac{7}{8}$ inch (9·53 to 22·23 mm.) thick, it was desirable for the sake of convenience that the test pieces for such plates should be of uniform width, and, in accordance with very general practice, a width of 2 inches (50·80 mm.) was selected. (2) With a test piece of a given form, the percentage of elongation was found to be less for thick plates than for thin ones; with steel of the same quality in other respects it was desirable therefore to choose widths of test piece which would be slightly in favour of the thicker plates. This is secured with the widths selected for the Standard Test Piece of form **A**.

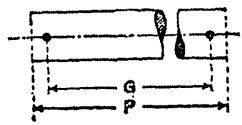
TEST PIECE **B**.



Gauge Length **G** to be not less than 8 times the diameter **D**. With enlarged ends :—Parallel Length **P** to be not less than 9 times the reduced diameter **D**.

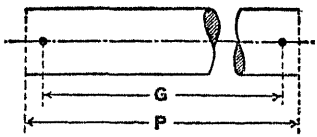
All test pieces of form **B** are strictly similar, and for the same material give the same percentage of elongation. They are nearly similar to a test piece of form **A**, 8 inches (203·20 mm.) in gauge length, 2 inches (50·80 mm.) wide and $\frac{3}{8}$ inch (9·53 mm.) thick.

TEST PIECE **C**.



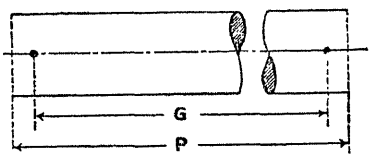
Gauge Length **G** = 2 inches (50·80 mm.).
Parallel Length **P** to be not less than $2\frac{1}{4}$ inches (57·15 mm.).
Dia. = 0·564 inch (14·33 mm.).
Area = $\frac{1}{4}$ sq. inch (161·29 mm.^{2}).

TEST PIECE **D.**



Gauge Length **G** = 3 inches (76·20 mm.).
Parallel Length **P** to be not less than $3\frac{3}{8}$ inches (85·72 mm.).
Dia. = 0·798 inch (20·27 mm.).
Area = $\frac{1}{2}$ sq. inch (322·58 mm.^{2}).

TEST PIECE **E.**

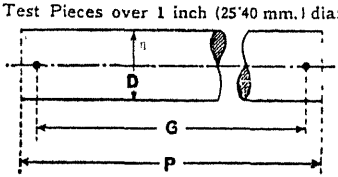


Gauge Length **G** = $3\frac{1}{2}$ inches (88·90 mm.).
Parallel Length **P** to be not less than 4 inches (101·60 mm.).
Dia. = 0·977 inch (24·82 mm.).
Area = $\frac{3}{4}$ sq. inch (483·87 mm.^{2}.).

Test pieces **C**, **D**, and **E** were arranged to meet the very common practice of making test pieces for forgings, axles, tyres, etc., of either $\frac{1}{4}$ square inch or $\frac{1}{2}$ square inch (161·29 or 322·58 mm.^{2}) in sectional area. With the gauge lengths decided upon, these three forms are very nearly similar, and, for a given material, give very approximately the same percentage of elongation. Though not exactly, they are approximately similar to the Standard Test Piece **F**, and for the same material give a nearly identical, but slightly greater, percentage of elongation.

TEST PIECE **F.**

(For Test Pieces over 1 inch (25·40 mm.) diameter.)



Gauge Length **G** to be not less than 4 times the diameter **D**.
With enlarged ends :—Parallel Length **P** to be not less than $4\frac{1}{2}$ times the reduced diameter **D**.

In some testing machines it was found inconvenient to use form **B** for bars of over 1 inch (25·40 mm.) in diameter, and form **F** of half the gauge length is designed to meet such cases. For a given material the percentage of elongation with test piece **F** is greater than with test piece **B**, and this difference is provided for in the British Standard Specifications.

Form of Ends

In the case of the round test pieces **B**, **C**, **D**, **E** and **F**, the form of the ends is to be as required in order to suit the various methods employed for gripping the test piece. When enlarged ends are used the length of the parallel portion of the test piece must in no case be less than that noted on the diagrams.

CHAPTER VIII

TIMBER

PLANTS are divided by botanists into two divisions : the Phanerogams and Cryptogams, the flowering and non-flowering plants.

It is the wood of the former division that is used for the arts. The phanerogams are divided into two classes, viz., endogens and exogens, the material from the latter only being used for building purposes.

On examining the stem of an exogenous tree, a section of which is shown in Fig. 48, four parts are distinctly noticeable, viz. :—

1. Pith.
2. Medullary Rays.
3. Annual Rings { Duramen. Alburnum.
4. Bark.

The pith is in the centre of the tree, and about this the annual rings are formed in concentric layers. The bark is that part exposed to the air, and serves as a skin to protect the newly-formed parts of the tree. The medullary rays occur as spider-like lines, radiating from the pith to the newly-formed wood.

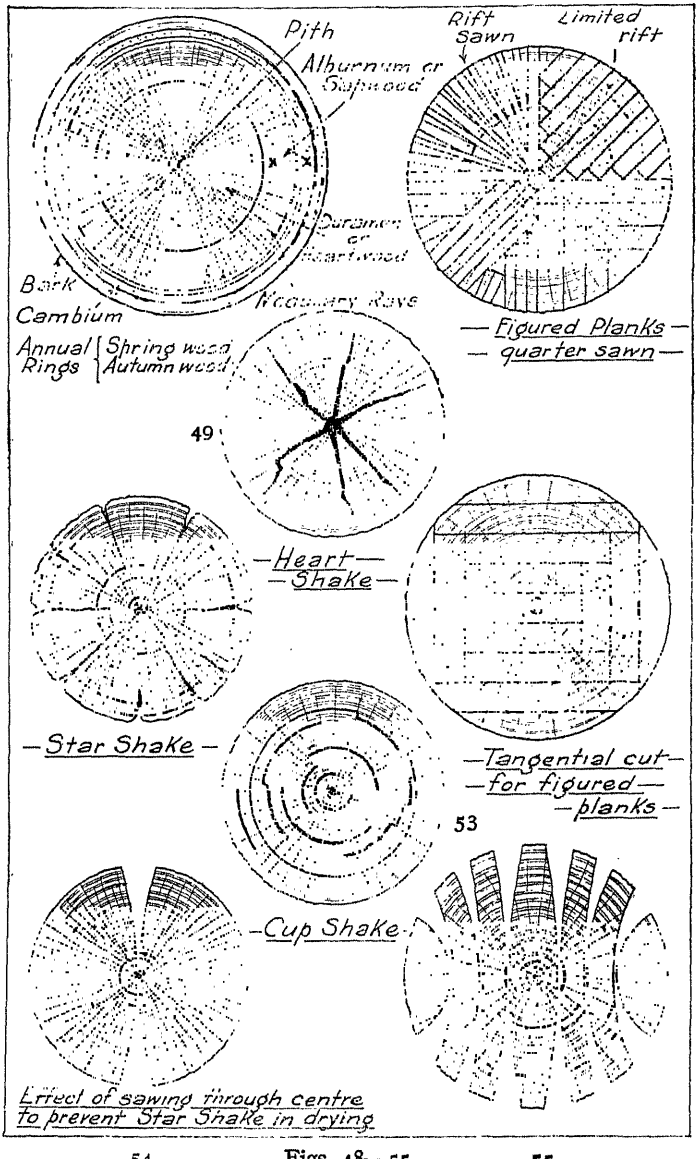
Pith or Medulla.—The pith is the first formed portion of the stem, and consists entirely of cellular tissue. When the plant is young the pith contains a large amount of fluid, which serves to nourish the various parts of the plant. On getting older, the carrying of the sap and moisture to the leaves, etc., is performed by the vessels of the woody fibre deposited about the pith, which now being of no further use, after a short time dries up and decays. The pith of all the branches is a prolongation of the pith of the main stem.

Medullary Rays.—The medullary sheath is a thin membrane composed of spiral vessels covering the pith, and lying between the latter and the first deposition of woody fibre. The medullary rays are vertical layers of a muriform cellular tissue, branching from the medulla and radiating from the latter to the most recently formed layers of sapwood. They serve to convey a portion of the descending sap to the vessels in the interior of the tree, and also the necessary air to combine with the substance of the sap and complete the formation of the various tissues. The medullary rays continue to lengthen and perform their function when the pith and the first-formed layers of woody fibre have commenced to decay.

Annual Rings.—The annual rings are formed of cellular tissue and woody fibre, arranged in concentric circles about the pith. These rings are called annual because one layer is added each year. This is true in the temperate climates, where the seasons differ in a marked manner, and where also there is one lot of leaves per year; but in the tropical climates, where the leaves are often shed twice in the same year, there would be a corresponding number of rings added.

The formation of the rings takes place as follows: In the spring a considerable quantity of moisture with other substances is absorbed by the tree from the soil; this is drawn up through the tubes of the alburnum or sapwood, and conducted from the roots to the extremities of the branches, where it helps in forming the leaves, from which a considerable quantity is evaporated, and a part turned into sap. At the same time the bark becomes loosened and a glutinous fluid called cambium is secreted between the bark and the last formed ring of sapwood; this is gradually converted into the cellular tissue of the next annual ring, and is known as the spring layer.

Towards the end of the summer, and during the autumn, the woody fibre commences to grow from the upper part downward, forming the dark ring in each annual layer. These fibres obtain their nutriment from the fluid of the cambium, and become attached to it; this is known as the autumn layer. During this time the moisture which has



48 49 50 51 52 53 54 55

Figs 48—55.

been turned into sap in the leaves descends in the outer layers of the tree, under the bark, being carried to the interior through the medullary rays, and so nourishing every part of the tree.

The first formed annual rings are gradually filled up with the substance carried in by the medullary rays and solidify, forming the duramen or heart-wood. In the living tree this is the first part to decay; decomposition commences at the pith and the oldest layers first, extending outwards. The heart-wood is the part used for constructional purposes, this being the most durable portion of the tree when converted.

The alburnum or sapwood should not be used for constructive or joinery work, as the juices and parts, not being properly hardened, readily decompose. The alburnum is easily distinguished from the duramen in the hard woods, being usually of a lighter colour, and in the softer woods the colours vary.

The annual rings in trees grown in hot countries are not, as a rule, so distinctly marked as those grown in a temperate climate, but gradually merge one into the other; where the seasons are more marked, the rings are more distinct.

Bark.—The bark consists of cells and woody fibre; a fresh layer is deposited each year on the interior. The bark is to a certain extent distensible, and increases slightly in girth as the diameter of the tree increases by the rising of the sap, but in the course of a few years the older layers become split, and the outer layers scale off.

In some specimens of exogens the bark becomes very thick and spongy, as is the case with those that supply cork.

Time for Felling.—Timber cannot be used immediately after felling on account of the moisture it contains.

A tree should be felled in its prime. If too young, there is too much sapwood; if too old, the wood is brittle and inelastic. Oak trees arrive at maturity in about 100 years, but they are often felled much before that age. The ash, elm, and larch should be felled when the trees are between

50 and 100 years old, and the poplar at between 30 and 50 years.

The spruce and pine in Norway are generally cut when between 70 and 100 years old.

Trees, on being felled, should be immediately stripped of their bark to allow of a more rapid evaporation.

The bark of the oak is valuable, and is generally stripped off when in its prime in the spring.

Logs should be cut longitudinally into two parts, as shown in Fig. 54, to the scantling required soon after felling, or if required to be stored in the log or balk should be floated to avoid large star shakes occurring through shrinkage.

Conversion.—If logs are left uncut after felling, the moisture dries out of the outer rings, which shrink, while the moisture still remains in the centre of the log and cannot shrink. The outer rings, not being distensible, develop star shakes. Therefore logs should be converted as soon as possible after felling to accelerate the seasoning by exposing a greater area of drying surface to the atmosphere, also to prevent the defects and distortion that result from the irregular rate of drying in the different parts of the log.

The conversion of the log facilitates the drying out of the moisture and causes shrinkage. Owing to the peculiar structure of timber, the shrinkage causes alteration in form varying with the method of cutting. In the uncut log shrinkage tends to take place in a circumferential direction, and if left would develop star shakes. Conversion avoids this. The log may be passed through a frame saw, which cuts it into a series of planks, a method adopted with hard woods in which the medullary rays are not visible and also with many of the conifers. The method known as rift sawing or radial cutting is adopted with oak and other timbers, in which the medullary rays are pronounced, or the planks may be quarter sawn or tangentially cut. These three methods are shown in Figs. 51–55.

The shrinkage in rift-sawn planks is only about one-half of that cut tangentially due to the restraining action of the medullary rays. There is, however, a considerable

amount of waste in the cutting; this is much reduced by the method of limited rift sawing shown in Fig. 51. In positions subject to abrasive action as in floors, the rift-sawn gives a harder wearing surface than quarter-sawn stuff.

Quarter-sawn stuff, in addition to shrinking more than rift-sawn, tends to cup or bend in a transverse direction (see Fig. 51). In woods with no strongly marked medullary rays, the finest figure is obtained by quarter sawing.

Timbers with a strongly marked grain are frequently cut for veneers in a circumferential direction by rotating the logs in a lathe against a fixed knife. The shrinkage of timbers in a longitudinal direction is negligible.

Timbers of the pine and spruce varieties (large quantities of which are imported into England from the countries around the Baltic) are usually converted before shipment, being sent to the market in the form of logs, balks, planks, whole deals, cut deals, battens, ends, masts, spars and poles, and prepared timber.

A log or stick is the trunk of a tree with the branches lopped off.

A balk is obtained by squaring a log.

Ends are short pieces of planks, deals, and battens under 8 feet long.

Masts have a circumference of more than 24 inches, spars and poles a circumference of less than 24 inches.

Boards are battens, deals, or planks under 3 inches thick.

Prepared timber is imported in scantling sizes, known as sawn timber, such as $4'' \times 2''$, $4'' \times 4''$, and many sizes used in building operations. A considerable quantity of wrought boarding also comes over in the form of weather-boarding, floor boards, matched and beaded boards, etc.

Classification of timber according to size :—

	in.		in.		in.		in.
Balk	12	by	12	to	18	by	18
Whole Timber	9	,,	9	,,	15	,,	15
Half	9	,,	4½	,,	18	,,	9
Scantling	6	,,	4	,,	12	,,	12
Quartering	2	,,	2	,,	6	,,	6
Planks	11	to	18	by	3	to	6
Deals			9	,,	2	,,	4½
Battens	4½	,,	7	,,	¾	,,	3
Strips and Laths	4	,,	4½	,,	½	,,	1½

Pieces larger than planks, generally called timber ; but sawn all round, called scantling ; when of equal dimensions, called die square.

Buying of Timber.—Pine and spruce timber is sold by the standard hundred, the load, or by the square of 100 feet super. There are several standard hundreds in use, as follows :—

London	...	120	pieces	12	ft.	long	9	in. by 3 in.
Petersburg	...	120	,,	6	,,	,,	11	,, ,, 3 ,,
Christiania	...	120	,,	11	,,	,,	9	,, ,, $1\frac{1}{4}$,,

The Petersburg standard is the one most generally followed, and equals 165 feet cube ; a load of timber is 50 cubic feet (hewn), so that there will be $3\frac{3}{10}$ loads in a standard. It will simplify calculation to commit to memory one or two facts relating to these measurements : 165 feet cube is 165 feet run of 12″ × 12″.

In dealing with a scantling of 12″ × 4″ its section is one-third of the 12″ × 12″, so that to make up a standard in that scantling it would require 165′ × 3′ = 495 feet run.

The price per foot run can be obtained in this way if the value of a standard is known, the cost of cutting being added.

Seasoning.—Newly felled timber contains a considerable quantity of moisture. Much of this is contained in the hollow spaces in the cells of the woody fibre. The remainder is absorbed in the tissue forming the cell walls. The water contained in the cells is readily given up, and when it is eliminated it has no effect in the way of expansion or contraction in the volume of the timber. The moisture absorbed by the cell walls takes a considerable time to evaporate, and it is the elimination of this moisture that causes the timber to shrink.

The art of seasoning is to extract the moisture as nearly as possible at a uniform rate from all parts of the timber, and to leave the residual moisture that cannot be extracted, uniformly distributed throughout the mass. Irregular drying is the cause of irregular shrinkage of the parts ; it sets up internal stresses between the fibres. When these are intense enough to overcome the cohesion of the fibres,

the result is the formation of shakes and warping, with the consequent waste and loss of strength in the material.

The extent to which timber can be dried naturally, depends upon the humidity of the atmosphere by which it is surrounded. The air contains moisture in the form of water vapour. The quantity depends upon the temperature, the higher the temperature the greater the quantity it can contain in suspension. The maximum quantity the air can contain at any temperature is termed the " saturation point." The actual quantity of moisture contained in the air at any time is generally given as a percentage of this maximum and is known as the " relative humidity."

Fibre Saturation Point.—In green timber, after the free water from the cells has been dried out, the amount of water remaining in the cell walls varies from 25 to 30 per cent. of the dry weight of the wood. This is known as the " fibre saturation point."

Moisture Content.—The actual moisture content of the wood is given as a percentage of the " oven-dry weight." The latter is obtained by subjecting a small sample to a temperature of 104° Cent. till its weight is constant. This is known as the " oven-dry weight." The moisture content is the excess weight of the specimen above the oven-dry weight. Timber in this country with a moisture content of 15 to 17 per cent. may be considered as air dried, and this percentage cannot be reduced with the average relative humidity prevailing in Britain. Timber with this percentage of moisture is sufficiently dry for all external joinery ; a percentage of 20 per cent. would be satisfactory for good carpentry work ; for external joinery from 15 to 17 per cent., and for internal joinery from 14 per cent. ; for halls and bedrooms 11 to 12 per cent. ; for living-rooms to 8 per cent. where in close proximity to radiators.

It is important to note that the moisture content of the timber bears a definite relation to the relative humidity of the atmosphere, and that if a piece of timber with a low moisture content be placed in an atmosphere of a high relative humidity, the wood will rapidly absorb moisture

and will soon reach the percentage moisture content that conforms to the relative humidity of the surrounding air. The average relative humidity in different years is a variable quantity.

If joinery is to stand up without any deterioration under any given conditions, its moisture content must approximate to that of the atmosphere in the position in which it is to be fixed, as the relative humidity of the latter is liable to vary within certain limits, and a timber with a low moisture content will quickly react to these changes. This means therefore that the relative humidity and temperature of the shops in which the joinery is prepared must closely approximate to those in the positions where the joinery will finally be fixed.

To determine the moisture content of the timber, a small sample should be weighed, first in its wet condition then dried as previously described, to determine its oven-dry weight. The percentage of moisture can then be found.

If W = wet weight
D = oven-dry weight
M = per cent. moisture content

Then $$M = \frac{W - D}{D} \times 100$$

Thus, if a sample be taken from a plank in a stack. Let the wet weight = 29 grammes and the dry weight = 25 grammes.

Then $$M = \frac{29 - 25}{25} \times 100$$
$$M = 16 \text{ per cent.}$$

This would represent average air-dried timber in Britain.

The practical application of this would be to determine when the timber in a stack is air dried and fit for, say, external joinery. Example: Take a stack of planks $12' \times 9'' \times 1''$. Select a plank and weigh; let its weight be 35 lbs. If a 16 per cent. moisture content is aimed at, then from the above formula find what should be the dry weight of the above planks.

Then $$M = 16 = \frac{35 - D}{D} \times 100$$

$$\therefore \quad D = \frac{3500}{116} = 30{\cdot}2 \text{ lbs.}$$

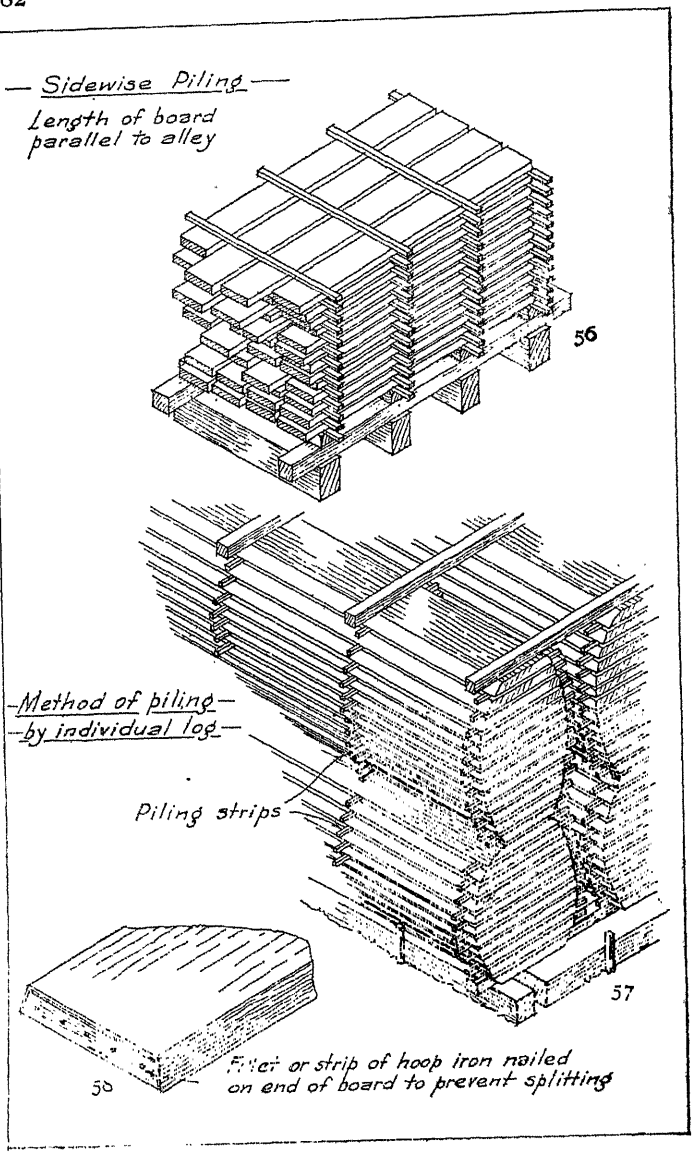
When, therefore, the planks weigh somewhere about 30 lbs. they would be ready for use.

Kiln Drying.—Timber to be employed for joinery in interiors where there is usually a higher temperature with a lower relative humidity than exists under external conditions, must have its moisture content reduced, which can only be done by subjecting it to a kiln-drying process. Great care must be exercised that the process is not applied too rapidly, or the external layers will quickly give up their moisture and tend to shrink, while the internal layers still remain full of moisture and unshrunk. The outer layers are thus stressed and may develop shakes if the tension becomes too intense. When the inner layers eventually give up their moisture and shrink, they tend to separate from the outer layers which are set, and again internal stresses develop, causing shakes.

The process employed is as follows: After the timber is stacked in the kiln with the greatest possible area exposed, the air is heated to a temperature slightly above 100° Fahr. and fully saturated; then it is circulated about the material. This serves the purpose of thoroughly heating the wood, without reducing its moisture content, thus avoiding evaporation from the outside layers, and therefore any shrinkage or tensional stresses. When the timber is thoroughly heated the humidity is gradually decreased and the temperature raised. As the movement of moisture can only take place slowly, the change must be gradually applied. The process is carried on until the humidity in the kiln conforms to the moisture content required in the timber, and the moisture remaining is evenly distributed throughout the mass.

Care must be taken after removal from the kiln, that the timber is not subjected for any length of time to an atmosphere with a higher relative humidity as the wood will rapidly reabsorb moisture.

Stacking.—Timber may be seasoned in the open or in sheds. In the first case, the stacks should have temporary roofs to protect them from rain. In the second, the sheds should be formed with louvres of a width sufficient to


— Sidewise Piling —
Length of board parallel to alley
56
Method of piling — by individual log —
Piling strips
57
58
Fillet or strip of hoop iron nailed on end of board to prevent splitting

Figs. 56—58.

protect the timber from a driving rain, and space should be arranged at the lower part of the wall to allow the air to circulate freely about the bottom of the stack. The ground should be drained, and preferably be concreted or paved to avoid the growth of weeds or fungi. The stacks should be arranged on sleepers placed at close intervals; these may be of wood or iron joists or rails, carefully levelled or graded, their upper edges all being in a perfectly plane surface. The sleepers should be supported by brick or concrete blocks, built at close intervals and not less than 1 foot in height.

There are many ways of stacking the timber. The best method is to place the planks in layers with a space of about 2 inches between them. Between the layers sticks are placed at right angles to the planks and from about 2 to 6 feet apart and having a thickness of from 1 to 3 inches, the spacing and thickness varying according to the thickness of the planks to be seasoned. The timbers and the sticks are placed immediately above those in the layers below them, to allow the air currents to circulate freely about the four surfaces of the timber (see Fig. 56).

The same method is employed for stacking balks of hard wood that have been cut into planks or boards (see Fig. 57). In the latter case the boards are usually of considerable width, and as the moisture is readily given up from the open end fibres, these boards if left will split, and these shakes will extend for a considerable distance along the planks. To avoid this defect, the ends of the boards are covered with strips of wood or metal to seal up the open end pores. They are also painted or the pores are sealed with special solutions. All of these methods have the object of preventing evaporation from this part (see Fig. 58).

Decay of Timber.—Timber, being an organic product, is liable to decay by the action of micro-organisms. Decay may also be due to mechanical processes, usually as a result of the timber being subjected to alternative wetness and dryness. This results in repeated expansions and contractions, which cause disruption of the fibres and chemical decomposition. This is usually known as wet rot. Timber

kept permanently wet or permanently dry has an indefinitely long life. Decay caused by parasitic or fungoid growths is termed dry rot, from the dry powdery condition of the timber.

Dry Rot.—There are several kinds of fungoid growths that attack timber. They differ in minor particulars, but in general their processes and effects on the timber are similar.

Timber that has been felled and allowed to remain in the forest is liable to be attacked by the *Polyporus vaporarius*. The first evidence of such infection is indicated by the presence of red stripes in the sawn wood. If such wood is thoroughly seasoned the mycelium present in the red stripes is killed. If the seasoning be neglected or imperfectly done, the mycelium, which possesses the power of remaining in a latent condition for some time, commences active growth when the wood is used in any part of a building where it is exposed to dampness, and this in some cases is unavoidable, as when the ends of joists are built into a wall.

Under such circumstances dry rot eventually appears.

On the other hand, the fungus is by no means rare on old beams and boards stored in woodyards, etc., and it is mainly from such sources that spores or portions of the spreading mycelium are introduced into buildings by new wood which has become infected.

The most malignant and common of these fungi is the *Merulius lacrymans*, the spores of which, floating about in the atmosphere, alight on timber when under favourable conditions, and these germinate, inserting their roots into the timber, the constituents of which they decompose, and so obtain their nutriment.

The fruit of the dry-rot fungus presents the appearance of irregularly shaped, flattened, or undulating patches of variable size, adhering by their entire under-surface to the substance to which they are growing. When mature the central portion of the patch is covered with an irregular network formed by slightly raised anastomosing ribs, and is of a rich brown colour, due to the enormous quantity of spores which are deposited on surrounding objects under

the form of snuff-coloured powder. These spores are diffused by currents of air, or by rats, mice, and insects.

The margin of the fruiting-patch is surrounded by a snow-white fringe of mycelium which spreads in every direction over surrounding objects, creeping up walls and passing through crevices, the advancing mycelium being supplied with food and moisture from the parent plant growing on wood.

This food is conducted through cord-like strands which form behind the thin advancing margin of mycelium.

Owing to this supply of food from a central source the mycelium can extend over stones and other substances not containing food, and thus spread from the basement to the top of a house. Each time the migrating mycelium comes in contact with wood the latter is attacked and a new centre of food-supply is established, from which strands spread in search of other sources of food. The mycelium often forms felt-like sheets of large size that can readily be removed intact. These sheets are white at first, but soon change to a pale grey colour.

The specific name of *lacrymans*, or " weeping," alludes to the power of the fungus to attract moisture from the atmosphere. Under certain conditions moisture is absorbed to such an extent that it hangs in drops, or even drips from the surface of the fungus. This moisture assists very materially in rotting the timber, which afterwards becomes quite dry and friable. Hence the popular name, " dry rot," which alludes to the last and most frequently observed stage of decay.

The fungus *Coniophora puteana* has many characteristics in common with the two preceding types. It requires a considerable amount of water and is found in very damp places.

There are several other of these dry-rot fungi, all of which require the following conditions to render them active : (1) Stagnant air, from which the necessary supply of oxygen can be drawn. (2) A temperature between about 50° and 70° Fahr. Temperatures above 90° Fahr. are fatal to their development and higher temperatures will destroy them. Lower temperatures will check them or cause them to become dormant. (3) Moisture is neces-

sary to their growth and support; therefore timbers, wherever possible, should be subjected to currents of fresh air, and all precautions should be taken to exclude dampness. Air currents and heat cannot always be provided, as in the ends of joists built into thin external walls; in such position some preservative should be used.

Properly seasoned timber should be employed, and care should be taken that adequate protection is provided on a building during erection to keep it under cover in the period immediately preceding fixing.

The chief protective measure that can be controlled by the constructor is to build in such a manner that dryness is ensured. The dry-rot fungus becomes inactive in timber with a moisture content under 18 per cent.; the fungus may, however, revive if the moisture content is increased beyond this percentage. Efficient damp courses in the ground story, especially in those parts in contact with the earth. Oversite concrete should be graded and of a mix that will render it impermeable to moisture. This should be coated with asphalt or tar in waterlogged situations. The undersides of floor boards, the plates and ground-floor joists should be treated with one of the well-known tar oil preservatives, and above all, a thorough circulation of air ensured by a plentiful supply of air bricks. All internal partition and sleeper walls should be honeycombed, and the space below the floors should be cleared of all rubbish, especially shavings or sawdust.

Where floor boards on ground floors are fixed to sleepers on concrete or wood blocks direct to it, a layer of asphalt or bituminous mastic should be provided between.

Insect Borers.—Timber, especially in old buildings, is frequently subjected to the ravages of insects, which bore into the wood and eventually destroy it. The chief among these are the *Lyctus brunneus*, the *Anobium punctatum* and the *Xestobium rufo villosum* or death watch beetle. The habits of these pests are very similar, and though their individual characteristics and differences may be of great interest to the entomologist, as far as the constructor is concerned their effects on timber are identical. The eggs are deposited in shakes, crevices or old bore holes in the

timber. The grubs, on emerging from the eggs, burrow into the timber. A short time after the chrysalis stage, the beetle sheds its pupal case and emerges from the wood, leaving a round hole.

The *Lyctus brunneus*.—The eggs of this order are hatched about February. The grub commences about March to bore and continues till October. It takes about a year for the beetles to emerge. As far as is known the *Lyctus* only attacks new wood, and confines its activities to the sap wood of hard woods. The coniferous woods seem to be immune from its attacks.

The *Anobium punctatum*.—This specimen, commonly known as the furniture beetle, hatches its eggs about September, and the grubs commence to tunnel along the grain. Between 1 and 2 years elapse before the beetle emerges from the chrysalis stage and makes its exit from the surface, generally during the months from June to August. The exit holes are about $\frac{1}{16}$ inch in diameter. They will apparently attack any wood, and after a few years reduce the material to a fine dust.

The *Xestobium rufo villosum* or death watch beetle, so called from the tapping noise made during the mating season, is the largest of the British wood borers. The eggs are hatched about September. The grubs immediately commence boring. They take from 1 to 2 years to reach the chrysalis stage. The beetles emerge from April to June, leaving a hole about $\frac{1}{8}$ inch in diameter. They attack chiefly the hard woods. Oak seems to suffer most from their ravages. This beetle confines its operations chiefly to old timbers, and may be looked for in the roofs and floors of old buildings where suitable conditions prevail. Well-seasoned timber is not as a rule attacked. The conditions favourable to the germination of these pests are dark, damp situations and a close unventilated atmosphere.

Preservatives.—There are a number of preservatives on the market of varying efficiencies which may be employed

as preventatives or for treating infected timbers. The timber is sprayed with various chemical compounds. Creosote or some of its derivatives seem to be the most effective. The objectionable odour emitted from these, however, usually debars them from use for internal work. Creosote must not be used on work that is to be painted or polished. For further and fuller information on timber seasoning, rot and preservation, reference should be made to the Reports of the Forest Products Research Laboratory.

Defects in Timber.—The following are the defects most common in timber :—

Cup Shakes, as shown in Fig. 53, separate the whole or part of one annual ring from another, and are caused by wind and frost in the growing tree.

Star Shakes, as shown in Fig. 50, radiate from the centre of the tree, increasing in width at the outside edge of the tree, and proportionate to the distance from the centre, due to the shrinkage of the timber in seasoning.

Heart Shakes, as shown in Fig. 49, are clefts or wide splits running right through the heart of a tree, and are certain signs of incipient decay at the centre.

Rind Galls are peculiar curved swellings, caused generally by the growth of layers over the wound remaining after a branch has been imperfectly lopped off.

Upsets are portions of the timber in which the fibres have been injured by crushing.

Foxiness is a yellow or red tinge caused by incipient decay.

Doatiness is a speckled stain found in beech, American oak, and other timbers. It is a disease producing local rot.

Twisted fibres are caused by the action of a prevalent wind turning the tree constantly in one direction. Timber thus injured is not fit for squaring, as so many fibres would be cut through.

Druxiness is the name given to decayed spots or streaks of whitish colour in timber.

Waney timber is the name given to cut timber showing at its angles that the rounded edges of the logs have been left on, so that the greatest possible rectangular section may be obtained with the least waste.

Characteristics of Good Timber.—In the same species that specimen will in general be the strongest and the most durable which has been the slowest in its growth, as shown by the narrowness of the annual rings. The cellular tissue of the medullary rays should be hard and compact, and when cut with a saw the woody fibres should not present a woolly appearance or clog the saw, but should appear firm and shining, emit its characteristic odour, and when struck give a clear ringing sound.

Amongst different species of trees the strongest timber is yielded by those flourishing in tropical climates, and amongst trees of the same species those grown in the cold climates.

The weights given for the various timbers, unless specifically stated, are for seasoned timbers, but in the same specie, the weights may vary considerably, and in the same piece of timber under ordinary conditions the weight will vary according to the state of the atmosphere.

The timbers in use for constructional and general work are classified under two heads—first, the needle-leaved or cone-bearing trees; second, the broad-leaf trees.

The first section includes the pines and spruces as follows:—

Northern Pine (Pinus sylvestris).—This timber is obtained largely from the following ports: Danzig, Memel; and Stettin in Prussia; Leningrad, Onega, Archangel, and Narva in Russia; Riga; Oslo and Dram in Norway; Gefle and Soderham in Sweden. It is also extensively grown in Great Britain, being known here as Scotch fir. It is of a light yellow colour; the annual rings are clearly defined, consisting of a light and a dark portion; they are regular and about $\frac{3}{16}$ inch in thickness; medullary rays not visible; straight grained; weighs about 36 lbs. per cubic foot; contains resinous substances which render it durable; it is strong and elastic; does not warp nor shake to any extent; is easy to work, and cuts clean and short. This wood is largely used for constructional work and for joinery, being suitable both for internal and external work.

American Yellow or White Pine (Pinus strobus).—This timber is exported from Quebec, St. John's and Shedac, and a few other Canadian ports. It is sometimes known as Weymouth pine. It is of a whitish or pale yellow colour, annual rings not very distinct; they are regular and about ¼ inch in thickness; medullary rays not visible; it is very straight grained; weighs about 29 lbs. per cubic foot; is not as strong nor elastic as northern pine; does not warp much, but is liable to shake; is very easy to work, and is used chiefly in joinery work for mouldings and wide panels; it does not prove durable when used externally.

This timber is exported in logs and deals. The first quality of deals is the "first bright"; "floated" and "dry floated" are inferior classes.

Brights consist of deals sawn from picked logs, and shipped straight from the saw mills.

Floated deals are floated in rafts down the rivers from the felling grounds to the shipping ports.

Dry floated deals are those which, after floating down, have been stacked and dried before shipment.

Red Pine (Pinus resinosa), exported from North America, and known as Canada Red Pine. The wood is white, tinged with yellow or straw colour, has a clean, fine grain, and works up to a surface having a smooth, silky lustre, weighs about 35 lbs. per cubic foot. Is used extensively for internal joinery, and is durable where well ventilated. Glue adheres well to the wood, and it is greatly used by cabinet-makers for veneering upon.

Sequoia Pine (Sequoia Sempervirens), from California, straight grained, may be obtained of great width and length, growing up to 400 feet high and 40 feet diameter, is easily worked; used for internal joinery. This wood is reputed to shrink in the direction of its length.

Kawrie Pine (Dammara Australis), from New Zealand, of yellowish-white colour; it possesses a silky straight grain; is generally free from defects; light, strong, and elastic, and is good for joinery, and weighs about 33 lbs. per cubic foot.

Pitch Pine (Pinus rigida).—This timber grows in the south-eastern States of North America, is shipped chiefly from the ports of Savannah and Pensacola. It is of a dark yellow or light reddish-brown tint; annual rings clearly defined and of a uniform width of about $\frac{3}{16}$ inch; medullary rays not visible; it is straight grained, and can be obtained in great lengths; is highly charged with resinous substances, rendering it very durable; weighs about 42 lbs. per cubic foot; it is very strong and, compared with other pines, difficult to work; has a tendency to stick to the tools on account of its larger quantity of resin. It is subject to heart and cup shake, shrinks considerably in drying, and also tends to warp.

The straightness of grain, great strength, and large scantling render it valuable for constructional work; it is largely used for piles; also for ornamental joinery work on account of the beautiful figure of its grain, although its excessive shrinkage renders it unsuitable for this purpose.

Douglas Fir.—This timber, which is known as Oregon or British Columbian Pine, is exported from the Pacific ports of North America. It is hard, strong, reddish-white in colour, close, straight and regular in grain, and similar in appearance to Pitch Pine. It is largely imported to England sawn into planks 4 to 8 inches thick, 12 to 24 inches wide, 12 to 40 feet long. The ordinary quality takes the place of Baltic Northern Pine for the purposes of carpentry and the higher grades are used for joinery. It is noted for its freedom from sap, shakes and knots. It weighs about 38 lbs. per cubic foot.

White Fir or Spruce (Abies excelsa).—This wood is obtained chiefly from the following ports: Onega, Narva, and Leningrad in Russia; Oslo, Dram, and Frederikstad in Norway; and Gothenburg, Sandsvall, and Hernosand in Sweden. It is of a whitish or very pale yellow colour; annual rings clearly defined, uniform, and about $\frac{3}{16}$ inch in thickness; medullary rays not visible. The wood is usually straight grained, weighs about 32 lbs. per cubic foot, contains resinous substances, but not to the same extent as northern pine. It is strong and elastic;

it warps and splits in drying; is tough but easy to work when free from knots, which in this wood are very hard; it cuts clean and free with the saw, and finishes with a silky lustre from the plane, and is sufficiently close grained to take a polish. The best kinds of this wood are used largely for internal joinery work; it is not durable when used externally; the coarser varieties are used for packing-cases, and for similar rough purposes. It has a beautiful figure, and is often varnished in order to enhance its appearance. It is much used for constructional work, but is neither as strong nor as durable as northern pine.

Cedar (Abies Cedrus).—From Asia and America. Is of a reddish-brown colour, porous, soft, and of light weight; has a pleasant odour, which is, however, obnoxious to insects and vermin, and is therefore suitable for furniture. It works easily, shrinks little, and is used for patterns, carved toys, pencils and boat-building, and weighs about 28 lbs. per cubic foot.

Larch (Genus Larix).—From Europe and America. Of honey-yellow or brownish-white colour, the toughest and most lasting of the coniferous order; has straight grain and is free from knots, but is very liable to warp, shrinks very much, and is extensively used for posts and railway sleepers. It weighs about 40 lbs. per cubic foot.

The second section includes oak, mahogany, etc.

Oak (Quercus).—The timber abounds in Europe (including Great Britain), Asia, and America. There are a great many species of oak, but all have the same general characteristics, differing only in minor details.

There are two kinds native to this country, viz., *Q. pedunculata* and *Q. sessiliflora;* the chief difference between the two lies in the arrangement of the flowers and leaves.

The former is generally supposed to be the more durable; the latter is credited with being tougher and more difficult to rend, and can be obtained in greater lengths, and is straighter grained than the pedunculata; it is light brown in colour; annual rings distinct and generally fairly

uniform, about $\frac{1}{8}$ inch in thickness; medullary rays strongly marked; grain fairly straight, but in trees grown in the open usually gnarled and twisted; weighs about 51 lbs. per cubic foot. It contains gallic acid, which rapidly corrodes ironwork, thus preventing the general employment of these two materials together; it is subject to warping and shaking; is very tough and difficult to work, but will take a high finish. It is greatly prized for ornamental joinery work on account of its figure and the beautiful markings of the medullary rays when the log is cut lengthwise radially; it is very durable and strong, and therefore valuable for heavy constructional work; it is very durable in either a wet or a dry situation, and proves more durable than most other woods in an alternately wet and dry position.

Baltic Oak is inferior in quality to the English, and is generally of a straighter grain. It is imported in logs 10 to 16 inches square, and planks 2 to 8 inches thick, and weighs about 53 lbs. per cubic foot.

American white oak (Quercus alba) can be obtained in larger sizes than any of the other kinds, and is not supposed to shrink as much in seasoning; it is straight grained, but not so durable as the English oak, and weighs about 62 lbs. per cubic foot.

Fumigation of Oak.—Oak is fumigated and darkened by subjecting it to the fumes of ammonia (NH_3) in an airtight cupboard or a fumigating chamber for a period of time according to the depth of the desired tint. The time for withdrawal may be determined by observation if the cupboard panels are of glass. Twenty-four hours is often required for the process.

Mahogany (Swietenia mahagoni).—Mahogany is obtained from the West Indies and Central America, the chief supplies coming from Cuba and Honduras. Mahogany is of a reddish-brown colour; annual rings not very distinct, but uniform; medullary rays invisible; fairly straight grained, weight (Cuba) about 48 lbs. per cubic foot, (Honduras) about 35 lbs. per cubic foot. It is strong

but inclined to be brittle ; it warps, shrinks, and shakes very little ; it is hard, not very difficult to work, and is capable of receiving a high finish and a splendid polish.

The wood lasts well when used internally, but is not durable when employed for external purposes ; it is chiefly used for cabinet work and ornamental joinery, for shop fittings and internal finishings. Cuba or Spanish mahogany, as it is sometimes called, is darker and richer in colour than the Honduras, and has a more wavy grain than the latter, and shows when cut a beautiful figure ; this renders it very valuable for the highest classes of joinery work. It is harder and denser, but does not attain such large dimensions as the Honduras. The Cuba may be easily distinguished from the Honduras by a chalk-like substance filling its pores. The Honduras is chiefly noted for the straightness of its grain, rendering it particularly adaptable for sticking mouldings. It is used for all kinds of internal joinery and cabinet work ; is largely used for pattern-making on account of the small amount of its shrinkage ; it is sometimes known as bay-mahogany or bay-wood.

Walnut.—First (Juglans regia), from Britain, unsuitable for beams, but used for ornamental joinery. It weighs about 44 lbs. per cubic foot.

Second (Juglans alba), the white walnut from North America, very tough and flexible, and weighs about 51 lbs. per cubic foot.

Third (Juglans nigra), from America, heavier, stronger, and more durable than European walnut. Not subject to the attack of worms, is of fine grain, and will polish well, and weighs about 57 lbs. per cubic foot.

Teak (Tectona grandis) is very durable wood for all work exposed to the weather ; it is exported from Burmah and other places. The exceptional straightness of grain renders it easy to work, but the fibres have a great tendency to split up in a longitudinal direction. It contains a resinous aromatic oil, which makes it very durable, and enables it to resist the white ant and worms, and tends to preserve iron fastenings ; it weighs about 49 lbs. per cubic foot.

Elm (Ulmus Campestris).—This timber is grown in large quantities in England. It is of a brown colour; annual rings distinct; medullary rays invisible to the eye; has a very twisted grain, not easily rent; weighs about 37 lbs. per cubic foot; it is very liable to warp and shake; is very tough and difficult to work; it is very durable when kept either thoroughly wet or perfectly dry. It is used chiefly for the sides and bottoms of carts, the hubs of wheels, for coffins, wood pulley-blocks, and for all similar purposes requiring a tough, strong wood.

Ash (Fraxinus).—This timber is obtained in large quantities in Great Britain. It is of a light brown colour; annual rings distinct; medullary rays not visible; is straight grained; weighs about 46 lbs. per cubic foot, and is very tough, strong, and elastic; is subject to shake in seasoning; durable if properly seasoned; has a large proportion of sapwood, but is very subject to the attack of worms. Its great elasticity debars its use for all large structural operations, but it is valuable as shafts for hammers, spokes for wheels, for oars, and in any position where it will be subject to sudden stresses.

Birch (Betula alba), from Europe and America. Light brown, hard, plain, and even in grain, is easily worked, but is neither strong nor durable, except for putlogs is not used for building purposes, but by chair-makers, cabinet-makers, and turners. It weighs when seasoned about 44 lbs. per cubic foot.

Beech (Fagus).—Large quantities of beech are obtained in England. It is of a light reddish-brown colour; annual rings distinct; medullary rays strongly marked, can be obtained in great lengths and is very straight grained; weighs about 45 lbs. per cubic foot; is strong, tough, and durable if kept dry; is close grained and easy to work, and will take a high finish. It is largely used for furniture, and owing to its close and even grain is valuable for tools which require an even wearing surface. Used for cogs in machinery wheels.

Basswood (Tilia Americana).—This wood is obtained from the United States and Canada. It is of a yellowish-white colour; annual rings indistinct; medullary rays invisible; is straight grained and of a uniform substance, soft and easy to work, and is of a very uniform grain and may be cut easily across or in any direction of the grain. It is largely used for cabinet work and also for carving.

Lime (Tilia Europæa).—Obtained plentifully in England. It is of a whitish colour; annual rings distinct; medullary rays invisible; very straight grained and uniform in density, very soft and easy to work in any direction. It is largely used for cabinet work and for fine carvings, and weighs about 33 lbs. per cubic foot.

Poplar (Genus Populus), of a yellowish or brownish-white colour, texture uniform. Light, soft, easily worked and carved, only indented, not splintered, by a blow. It should be well seasoned, and when kept dry is tolerably durable. It weighs about 33 lbs. per cubic foot.

Chestnut (Castanea vesca) from Europe and America, is of low growth, and of similar appearance to oak, but has no alburnum nor sapwood, and the medullary rays are not easily seen. It attains great dimensions, but is only used for common work, such as posts and rails, and weighs about 41 lbs. per cubic foot.

Hornbeam, from Britain. White colour, close grain, tough, hard, strong, and of moderate weight; is specially useful for cogs in machinery. It weighs about 52 lbs. per cubic foot.

Sycamore (Acer Pseudo-Platanus).—This timber abounds in England. The wood is sometimes known as the common or great maple. It is brownish or yellowish-white in colour; annual rings distinct; medullary rays small but distinct, often has a beautiful figure; fairly strong, and difficult to work; weighs about 38 lbs. per cubic foot, is durable when kept dry. It is used chiefly for furniture and ornamental joinery.

Maple (Ace Saccharinum).—This timber abounds in Canada. Its heartwood is light brown and its sapwood yellow in colour, weighs about 43·5 lbs. per cubic foot, does not easily splinter, and is largely used for wood block flooring. Cut tangentially to the annual rings as veneers, it gives the beautiful markings known as the bird's eye maple.

Jarrah (Eucalyptus marginata).—This wood, from Western Australia, is of a red colour, hard, heavy and close in texture, works smoothly and takes a good polish. It is somewhat brittle, resists well the sea worm and the white ant, and is good for positions such as posts partly under and partly above ground. It is largely used as wood paving blocks, also by cabinet makers; but, owing to its brittleness, is not good for constructional members to resist a transverse or a tensional stress. Its weight is about 63 lbs. per cubic foot.

Greenheart (Nectandra rodixi), a strong and durable timber from British Guiana. This timber resists the attacks of the sea worm, and ranks next to teak for resisting the attacks of the white ant, and is well adapted for planking vessels, piles, and structures under water. It requires careful working, being liable to splinter. It weighs about 62 lbs. per cubic foot.

The following are a few of the leading European ports exporting timber for the English market:—

Norway.—Oslo, Drammen, Frederikstad.

Sweden.—Gothenburg, Soderham, Gefle, Sunderswall, Stockholm.

Prussia.—Memel, Danzig, Stettin.

Russia.—Leningrad, Archangel, Riga, Onega.

American Ports.—Quebec (yellow deals), St John's (spruce), Richibucto, Shedac, Miramichio.

Expansion of Timber.—Expansion takes place along the

grain when dry, when the temperature is raised from 32° to 212° Fahr. :—

Baywood	...	...	0·000461 to 0·000566
Deal	...	...	0·000428 ,, 0·000438

The tabulated values given above represent the average of a large number of specimens. It should be remembered that different specimens of the same kind of timber give widely different results, and in practice care must be taken to ascertain that the specimens are up to good average quality and free from defects. The tensional value of timber is difficult to obtain with any great degree of accuracy, therefore bars to be subjected to tension should be selected straight in grain, free from any large loose or dead knots, and the factor of safety employed for permanent work should never be less than one-eighth. For the modulus of rupture for members under transverse stress and compressional members a factor of safety of one-fifth is sufficient.

MECHANICAL AND PHYSICAL PROPERTIES OF TIMBER. FOREST PRODUCTS RESEARCH, PRINCES RISBOROUGH, SEPT. 1929.

Species Common and Botanical Names	Locality where Grown	Rings per inch	Proportion of Summerwood	Moisture Content	Specific Gravity	Weight per Cubic Foot	Shrinkage			Static Bending						Impact Bending				Compression Parallel to Grain			Compn. Perpr. to Grain	Hardness			Shearing Parallel to Grain		Cleavage		Tension Perpendicular to Grain	
							Green to air dry* based on green dimensions						Work in bending per cubic inch of specimen											Load to imbed a 0·444-in dia. steel ball to one-half of its diameter			The plane of failure being		Splitting Strength per inch of width of specimen 3 in. long, the plane of failure being		Apparent Average Tensile Strength, the plane of failure being	
					Based on Weight when oven-dry and on Volume as tested	As tested	Volumetric	Linear		Fibre Stress at Limit of Proportionality	Equivalent Fibre Stress at Maximum Load	Modulus of Elasticity (apparent)	To Limit of Proportionality	To Maximum Load	Total	Fibre Stress at Limit of Proportionality	Modulus of elasticity (apparent)	Work in Bending to Limit of Proportionality per cubic inch of Specimen	Height of drop of 50-lb. Hammer causing complete failure of Specimen	Compressive Stress at Limit of Proportionality	Maximum Crushing Strength	Modulus of Elasticity	Fibre Stress at Limit of Proportionality	Radial Surface	Tangential Surface	End Surface	Radial	Tangential	Radial	Tangential	Radial	Tangential
								Radial	Tangential																							
			Per cent.	Per cent.		Lb.	Per cent.	Per cent.	Per cent.	Lb. per sq. inch	Lb. per sq. inch	1,000 lb. per sq. inch	Inch lb.	Inch lb.	Inch lb	Lb. per sq. inch	1,000 lb. per sq. inch	Inch lb.	Inches	Lb. per sq. inch	Lb. per sq. inch	1,000 lb. per sq. inch	Lb. per sq. inch	Lb.	Lb.	Lb.	Lb. per sq. inch	Lb. per sq. inch	Lb.	Lb.	Lb. per sq. inch	Lb. per sq. inch
1	2	4	5	6	7	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34	35	36
TESTED IN A GREEN CONDITION																																
Ash (*Fraxinus excelsior*)	Coleford, Gloucester	9	66	49·9	0·546	51·0	13·0	3·2	9·0	4,180	8,680	1,506	0·66	18·7	49·0	11,980	1,777	4·57	50	2,440	3,690	1,656	793	859	915	1,060	1,270	1,363	440	504	959	1,115
Beech (*Fagus sylvatica*)	Forest of Dean, Glos	8	34	92·1	0·563	67·4	12·4	3·5	8·3	3,870	8,200	1,366	0·63	12·1	24·6	10,150	1,885	3·04	36	2,270	3,510	1,658	615	1,110	1,103	1,129	1,306	1,580	393	562	811	1,262
Elm, Common (*Ulmus campestris*)	Wimborne, Dorset	8	66	141·4	0·437	65·7	9·3	3·0	5·6	2,730	5,430	758	0·55	7·4	10·4	7,920	1,105	3·18	24	1,350	2,360	842	509	838	830	848	1,203	1,310	345	369	682	830
[illegible]	Watlington, Oxfordshire	9	63	145·6	0·444	68·0	8·0	3·0	5·0	2,470	5,440	801	0·45	7·3	16·4	8,090	1,301	2·79	27	1,220	2,470	932	504	899	897	905	992	1,065	326	385	688	846
[illegible]	Wimborne, Dorset	7	63	152·3	0·426	67·1	9·6	3·3	5·3	2,910	5,610	779	0·62	8·1	16·2	7,420	981	3·11	27	1,460	2,450	827	555	834	826	825	1,117	1,199	313	369	659	826
[illegible]	Wadhurst, Sussex	11	59	90·4	0·555	65·9	10·3	3·7	7·3	4,470	9,280	1,329	0·86	9·2	23·3	9,690	1,767	2·99	33	2,910	4,090	1,650	657	1,206	1,194	1,215	1,150	1,306	389	446	802	1,000
[illegible]	Taymount, Scotland	6	38	42·7	0·418	37·2	5·2	2·4	4·8	3,910	7,440	1,233	0·70	9·3	21·2	9,630	1,787	2·90	32	2,260	2,830	1,414	427	428	431	625	892	1,000	219	259	431	557
[illegible]	Argyllshire, Scotland	4	44	60·9	0·407	40·9	6·6	2·6	4·5	4,660	7,660	1,408	0·88	7·6	26·8	9,940	1,723	3·21	25	2,030	3,260	1,455	419	642	640	688	958	1,050	219	258	434	538
[illegible]	Forest of Dean, Glos.	8	31	56·1	0·483	47·0	5·6	1·8	4·3	3,120†	7,110†	1,219†	0·46†	10·0†	40·3†	9,580†	1,560†	3·26†	46†	2,520	3,640	1,387	476	465	467	597	943	905	197	227	352	489
[illegible]	Argyllshire, Scotland	10	41	48·4	0·501	46·3	—	—	—	3,150	8,040	1,389	0·44	10·3	31·8	10,500	1,948	3·23	36	1,470	3,900	1,444	521	720	699	714	1,022	996	243	247	439	524
[illegible]	Wells, Norfolk	8	35	128·1	0·410	58·4	7·1	2·2	5·0	2,910	5,380	1,119	0·44	6·1	13·6	7,580	1,435	2·25	19	1,780	2,530	1,170	305	376	317	388	819	882	187	226	408	545
[illegible]	Knightswood, Hants.	7	36	127·2	0·448	69·4	8·2	3·4	6·3	4,100	7,040	1,457	0·65	7·2	23·0	9,690	1,774	3·02	26	2,170	3,210	1,535	360	517	514	514	800	810	224	230	376	456
[illegible]	Bedgebury, Kent..	10	34	93·3	0·467	56·2	7·5	3·7	5·0	3,510	5,940	1,325	0·52	5·1	34·2	9,730	1,780	2·98	28	2,020	2,990	1,472	381	485	380	456	778	805	213	247	392	556
[illegible]	Loch Tay, Scotland	12	41	74·8	0·428	46·7	8·3	2·8	5·6	3,080	5,490	1,122	0·48	5·1	20·0	9,600	1,521	3·43	24	1,956	2,720	1,319	349	516	515	509	846	848	203	232	343	425

* Air-dry for these tests corresponds to a moisture-content of 12 per cent

TESTED IN A SEASONED CONDITION

Ash (*Fraxinus excelsior*)	Coleford, Gloucester	9	59	13·7	0·621	44·0	—	—	—	6,060	13,200	1,761	1·17	9·6	21·8	17,630	2,313	7·12	46	3,160	6,080	2,129	1,130	1,088	1,645	1,698	2,049	2,196	702	661	1,236	1,377
Elm, Common (*Ulmus campestris*)	Wimborne, Dorset	9	66	12·2	0·468	32·8	—	—	—	3,610†	9,160†	1,180†	0·62†	8·3†	15·5†	9,740†	1,845†	2·95†	23†	1,660	4,320	1,152	754	769	791	969	1,667	1,790	280	348	506	736
Elm, Common (*Ulmus campestris*)	Watlington, Oxon.	10	61	12·4	0·471	33·0	—	—	—	4,420†	8,180†	891†	1·22†	7·6†	16·0†	—	—	—	—	1,960	4,900	1,227	688	933	983	1,095	1,629	1,679	395	430	772	1,031
[illegible]	Wimborne, Dorset	8	67	10·5	0·460	31·7	—	—	—	6,120	11,470	1,303	1·60	11·2	20·5	11,490†	2,178†	3·41†	25†	1,980	3,600	1,136	694	891	898	1,075	1,564	1,716	289	367	595	787
[illegible]	Wadhurst, Sussex	12	60	12·3	0·575	40·2	—	—	—	6,860	12,950	1,645	1·20	11·2	20·7	13,330	2,850	3·47	30	3,100	7,250	1,950	867	1,458	1,421	1,389	1,525	2,271	313	393	768	920
[illegible]	Taymount, Scotland	6	46	13·2	0·429	30·2	—	—	—	5,910	10,930	1,488	1·32	9·8	32·7	10,220	1,954	3·03	31	3,780	5,890	1,730	818	900	893	976	1,390	1,310	227	264	406	517
Douglas fir (*Pseudotsuga Douglasii*)	Argyllshire, Scotland	4	43	12·1	0·431	30·1	—	—	—	5,450	12,240	1,803	0·94	9·6	17·1	12,560	2,901	3·05	27	3,180	6,270	1,642	792	662	707	987	1,289	1,366	217	252	257	411
Larch, European (*Larix europaea*)	Forest of Dean, Glos.	8	37	13·5	0·503	35·6	—	—	—	5,700	11,350	1,387	1·31	14·9	27·3	9,690†	1,798†	2·94†	33†	3,400	6,370	1,774	782	987	982	1,102	1,457	1,454	253	267	394	394
Pine, Corsican (*Pinus laricio*)	Wells, Norfolk	7	39	13·5	0·441	31·2	—	—	—	5,570	9,690	1,399	1·25	7·4	13·6	11,380	2,596	2·65	23	3,030	5,070	1,548	598	710	703	789	1,228	1,404	221	295	443	630
Pine, Corsican (*Pinus laricio*)	Knightswood, Hants	7	38	11·2	0·463	32·1	—	—	—	6,230	13,620	1,998	1·15	11·9	22·4	13,470	2,846	3·89	26	3,510	6,900	1,990	695	829	892	892	1,440	1,423	240	289	485	552
Pine, Scots (*Pinus sylvestris*)	Bedgebury, Kent.	9	32	12·7	0·484	34·0	—	—	—	6,670	11,700	1,602	1·57	10·7	21·1	16,170	2,282	6·33	20	3,240	6,250	1,654	747	857	850	845	1,414	1,510	257	304	437	589

CHANGE IN PROPERTY DUE TO SEASONING. *"Air-dry" values as percentages of "green" values*

Ash	—	—	—	—	113·7	86·3	—	—	—	145·0	152·1	116·9	177·3	51·3	45·4	147·2	130·2	155·8	92·0	129·5	164·8	128·6	142·5	196·5	179·8	160·2	161·3	161·1	159·5	137·1	128·9	123·5
Elm, Common	Wimborne	—	—	—	107·1	49·9	—	—	—	132·2	168·7	155·7	112·7	112·2	149·0	123·0	167·0	92·8	95·8	123·0	183·0	136·8	148·1	91·8	95·3	114·3	138·6	132·1	75·4	94·3	74·2	88·7
Elm, Common	Watlington	—	—	—	106·1	48·5	—	—	—	178·9	150·4	111·2	271·1	104·1	97·6	—	—	—	—	160·6	198·4	131·6	136·5	103·8	109·6	121·0	164·2	144·1	121·2	111·7	112·2	121·9
Elm, Dutch	Wimborne	—	—	—	108·0	47·2	—	—	—	210·3	204·4	167·3	258·1	138·3	126·5	154·8	219·8	109·6	92·6	135·6	146·9	137·4	125·0	106·8	108·7	130·3	140·0	143·1	92·3	99·4	90·3	95·3
Oak	—	—	—	—	103·8	61·9	—	—	—	131·1	156·4	123·8	139·5	121·7	88·8	137·6	161·3	116·0	90·9	106·5	177·3	118·2	132·0	120·9	119·0	114·3	158·7	173·9	80·5	88·1	95·8	92·0
Douglas fir	Taymount	—	—	—	102·6	81·2	—	—	—	151·1	146·9	120·7	188·6	105·4	154·2	106·1	108·7	104·5	96·9	167·2	211·7	122·3	191·6	210·3	207·2	156·2	155·8	131·0	103·6	101·9	94·2	92·8
Douglas fir	Argyllshire	—	—	—	105·9	73·6	—	—	—	116·9	159·8	128·0	106·8	126·3	63·8	126·3	168·4	95·0	108·0	156·6	192·3	112·8	189·0	103·1	110·5	143·4	134·5	130·1	99·1	97·7	59·2	78·4
Larch, European	—	—	—	—	104·1	75·7	—	—	—	182·7	159·6	103·8	284·8	149·0	67·7	101·1	115·2	90·2	71·7	134·9	175·0	127·9	164·3	212·2	210·3	184·6	154·5	160·7	128·4	117·6	111·9	80·6
Pine, Corsican	Wells	—	—	—	107·6	53·4	—	—	—	191·4	180·1	125·0	284·1	121·3	100·0	150·1	180·9	117·8	121·0	170·2	200·4	132·3	196·1	188·8	221·8	203·3	149·9	159·2	118·2	130·5	108·6	116·7
Pine, Corsican	Knightswood	—	—	—	103·3	50·6	—	—	—	151·9	193·8	137·1	176·9	165·3	97·4	139·0	160·4	118·9	100·0	161·7	214·9	129·6	193·0	160·3	173·5	173·5	180·0	175·7	107·1	125·6	123·7	121·0
Pine, Scots	Bedgebury	—	—	—	103·6	60·5	—	—	—	190·0	197·0	120·9	301·9	209·8	61·7	166·2	128·2	212·4	71·4	160·4	209·0	112·4	196·1	176·7	223·7	185·3	181·7	187·6	120·6	123·1	111·5	105·9

† These figures are based on less than five tests

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CHAPTER IX

PAINTS AND VARNISHES

Definition.—Paint is usually a coloured liquid laid on the surfaces of building materials by means of a brush, drying as an impervious coat to protect the covered material from the effects of the atmosphere, and also for decorative purposes.

Paints are composed of four or five components—the base, inert filler, vehicle, solvent, driers, to which is often added a colouring matter called a pigment.

Base.—There are several bases used for paints, many of them for special purposes. The following are the four most commonly used in building work: White lead, red lead, zinc white, and oxide of iron.

White Lead is a basic lead carbonate $2PbCO_3$, $Pb(OH)_2$, and is that mostly used as a base for painting all ordinary building work. It has a greater covering power than any other base, and weathers well; but it is liable to become discoloured by sulphuretted hydrogen (H_2S), and is very poisonous. It frequently contains a small amount of iron, which turns it to a yellow colour. There are two methods of manufacturing white lead: First, by placing sheets of lead (Pb) in tan, and subjecting them to the fumes of acetic acid (CH_3COOH); this has the effect of covering the sheets with a crust of the carbonate, which is removed and ground to a fine powder, and a fresh surface of the lead exposed to the fumes of the acetic acid; this is known as the Dutch method. Secondly, by dissolving lead monoxide (PbO) in acetic acid, and then forcing carbon dioxide (CO_2) contained in the smoke of a coke fire with chalk through the lead acetate, $Pb(CH_3COO)_2$.

White lead may be obtained as a powder or mixed with about 8 per cent. of linseed oil.

White Lead should be covered, as exposure to the air turns it grey, and should be kept a considerable time before using. If used too fresh, it acquires a yellowish tinge.

Red Lead, or minium, is tri-plumbic tetroxide (Pb_3O_4), used chiefly as a priming coat for wood, and also as a base for some red paints. It is not durable if exposed to acids, foul air, or metallic salts; other oxides of lead and white lead alter its shade of colour. It is prepared by heating lead in an open furnace, thus forming litharge (PbO), and then heating it a second time, when it takes up more oxygen, forming Pb_3O_4.

Zinc White.—Of late years the use of zinc white, which is an oxide of zinc (ZnO), has been rapidly extending as a base for paints for the following reasons: its colour is not visibly changed by the sulphur impurities in town atmospheres nor by the sea air, its covering power and opacity in the second and following coats are equal to that of lead oxide paints, it has not the objectionable poisonous properties of white lead, for which reason the latter, as a base, is forbidden to be used in France and other countries; it may be tinted with ordinary pigments, and the cost, where judiciously used, is less than that of white lead. Zinc oxide is prepared by burning zinc in a retort, through which a current of atmospheric air is passed. The zinc oxide passes into a receptacle, in which it is collected and compressed to make it more dense.

Magnetic Oxide of Iron (Fe_3O_4) is used as a base in paints chiefly for covering iron work. It is supposed to be better for iron work, as no galvanic action can be set up between the base of the paint and the metal to be covered. It is prepared by roasting and grinding a brown hæmatite ore, consisting principally of iron oxide and silica.

Inert Fillers.—The bases of paints are diluted or extended with inert material called fillers, for the purpose

of correcting the weight of the paint, making it lighter or heavier as may be desired, increasing the durability, and to lessen the cost. These are sometimes termed adulterants, but as yet there is no standard specification. The following give some of the principal fillers: Barytes, silica, silicates of magnesia, silicates of alumina, calcium carbonate, whiting, gypsum, charcoal. For masonry the natural fillers are to be preferred; several are equally good for steel, and probably silica or barytes for timber. No single filler is best for all purposes.

Vehicle.—A vehicle is a liquid of a drying nature, capable of dissolving and holding bases and pigments in suspension, and to enable the paint to be laid on in thin and uniform coats, and to enter the pores if it be a porous material, where it hardens, and thus forms a durable and impervious skin. For oil colours, such as the ordinary paint, the vehicles are oils, but for whitewash or distemper water is used.

The vehicles chiefly used for paint are linseed, poppy, and nut oils, which belong to the class known as fixed oils.

Linseed Oil is most commonly used for all ordinary work; it is obtained from the flax plant by crushing the seed, heating it and forcing the oil out in hydraulic presses; it is then allowed to stand, and the clear oil is drawn off. This is known as raw oil; it is transparent, with a slight amber tint, and improves in drying and colouring properties by keeping several years. It is useful for delicate tints, and therefore for internal work. Linseed oil is often boiled before use by heating it to about 90° with drying substances, such as red lead and litharge, and then raising the temperature to about 200°, at which it is kept for 3 or 4 hours, when it is allowed to stand, and the albuminous matter it contains settles and is then separated. Boiled oil is much thicker and darker than the raw oil; it dries more rapidly, and has a greater body, is more durable, and is therefore more suitable for external work.

Poppy Oil is obtained by pressing poppy seed. It is often used for delicate colours, being clearer than the

linseed oil, but it is inferior to it in drying and tenacious properties.

Nut Oils are used by painters because of their cheapness, but as they are inferior in every respect to both linseed and poppy oil their use is limited to inferior work.

Solvent.—Turps is added to paint primarily as a solvent, and also to dilute it to work more freely.

Turps or Oil of Turpentine ($C_{10}H_{16}$) is a volatile oil prepared by distilling turpentine, a resinous substance, obtained by tapping trees of the coniferous order. It dries partly by evaporation and partly by the absorption of oxygen, the result being a resinous body.

It is used with the base without the oil when the glossy surface left by the oil colour is not required ; a coating of such colour is known as a flatting coat.

Driers.—Linseed oil dries by the absorption of oxygen ; this may be greatly accelerated by adding substances containing a large proportion of that element. Driers are supposed to act either by parting with some of their oxygen or by enabling the oil in some way to combine with the oxygen of the air, the latter being the more probable.

The following are some of the substances used for this purpose : Litharge, lead acetate, zinc sulphate, manganese dioxide, and red lead.

Litharge (PbO), an oxide of lead, is most commonly used. Massicot, a superior kind of litharge, is prepared by heating the lead to a degree just insufficient to fuse the oxide.

Lead Acetate.—$(CH_3COO)_2$ Pb, ground in oil, is used as driers for the lighter tints.

Red Lead.—Pb_3O_4, which is lead oxide, is often used as driers when its colour does not affect the tint, but it is less powerful in its action than litharge.

Manganese Dioxide.—MnO_2, is quick and powerful in its action, but can only be used for the deep tints, as it is of a dark colour.

Zinc Sulphate.—$ZnSO_4$ and *Manganese Sulphate* ($MnSO_4$) are used as driers for zinc paints. No driers containing lead should be used for a paint with a zinc base, as a voltaic action would be set up.

Driers should not be used with pigments that dry well, nor in excess, as this retards the action of drying; they should not be added till the colour is about to be used.

Terebene is a solution of one of the driers in oil of turpentine. It is used in paints that are required to dry quickly.

Pigments are colouring matters finely ground, used to give opacity and colour to paint for ornamental purposes. They are prepared from earthy, animal, and metallic substances in two forms, either as a finely ground powder, mostly used for tinting distemper, or ground in oil for tinting oil paints.

Generally pigments of earthy or animal origin are less permanent than mineral colours.

In estimating painters' work the cost will largely depend upon the pigment used, some pigments being more expensive to produce than others. For convenience they are usually classified under three heads, viz. :—

Common Colours, which comprise such as lampblack, red lead, white lead, Venetian red, greys, ochres, and umbers.

Superior Colours, blues, warm tints, light yellows, mineral greens.

Delicate Tints, pea green, verditer, bright blues, rich reds and pinks.

Knotting.—This as used consists of shellac dissolved in naphtha, and is known as patent knotting. It is used to cover knots in wood to prevent any exudation of turpentine, or any marks to show through the paint caused by the absorption of the knots.

Mixed Paints.—Mixtures including more than one of the bases, such as white lead or zinc oxide, are known as mixed

paints. These have been, of late years, extensively used in America and England. White lead bases dry soft; zinc white dries hard, and it is contended that a certain blend of the two will give better practical results, for most purposes, than either used separately.

Inert fillers, such as barytes, silica, silicates of magnesia, silicates of alumina, calcium carbonate, whiting, gypsum, charcoal and others, usually looked upon as adulterants, have been shown by Max Toch, if scientifically made and used in proper proportion, to outlast other mixtures, and to be superior and cheaper; thus he instances a case of a mixture of one-third carbonate of lead, one-third zinc oxide, and one-third barytes on an exposed wall of a high building in New York City, which was, after 20 years, in a better state of preservation than a wall painted by the Dutch process, white lead, was, after 5 years. This established that the great increase in the life of the paint was due to the inert filler, as it is conceded that no paint is supposed to last 20 years. Another case is given by the same authority, that on the coast of Maine it was found that pure white lead would not stand the exposure more than a year, at the end of which time it resembled whitewash, and presented a poor surface for repainting. A mixture was made at the same time of one-third silica—prepared by heating and washing—one-third zinc oxide, and one-third white lead. These materials were ground together in pure linseed oil, and sufficient driers added. At the end of 7 years this paint was still in good condition, and presented an excellent surface for repainting.

Ready Mixed Paints.—The manufacture of mixed paints has of late assumed considerable dimensions, due partly to important manufacturers employing expert chemists to test and select suitable ingredients, and to the use of specially constructed machinery suitably arranged, for the intimate mixing, efficient grinding, cooling, tint correcting, straining, and packing of the paint ready for use, which operations, especially those of thorough mixing and extreme fineness in grinding, are so much more perfect and reliable than when executed by the usual hand method; and it is very probable that mixed paints would be even

more largely employed if manufacturers would state plainly, as suggested by Toch, the names of the bases, inert fillers, vehicles, solvents, and driers that they are selling in the compound mixture, also the date of mixing.

Tar is a substance obtained by the distillation of the wood of pine trees, and also in the distillation of coal for the manufacture of gas, the tar being a by-product; it is often used for forming a paint for preservative purposes only. The tar is mixed with turps and linseed oil or slack lime. A small quantity of pitch, which is obtained by distilling tar, is added in hot weather to prevent the tar from running; a little lime answers the same purpose.

Creosote is a product obtained by distilling tar, and is a largely used and effective preservative paint for wood.

Solignum, a wood stain, is manufactured by Major & Co., tar refiners, is an excellent preservative for timber and is used in many parts of the world where timber is subject to dry rot. It is made in several shades—brown, green, red and yellow.

Fire-resisting Solutions.—Asbestos paint possesses the valuable property of retarding the action of fire, and for that reason has been largely adopted for public buildings.

Coatings of sodium tungstate also retard the action of fire, and have in some instances resisted the action of fierce fires for as long as 20 minutes shortly after application.

Wherever timber work in fire-resisting constructions is accessible, it may with great advantage be coated with either of these solutions.

VARNISHES

Varnish is a solution of fossil or semi-fossil resins dissolved either in linseed oil or spirits. It is a liquid substance which, spread over solid bodies, gives a brilliant, transparent and durable covering. It is used to preserve painted work exposed to the weather, and also to improve the appearance by covering it with a shiny transparent coat of resin.

It is applied to wallpapers and also to joinery work in woods with a beautifully marked grain, to preserve and improve their appearance.

There are several kinds of resins used for making varnish, the three principal being amber, gum animé, and copal; the latter two are those chiefly used for building work, and the chief supply is the Kauri from New Zealand.

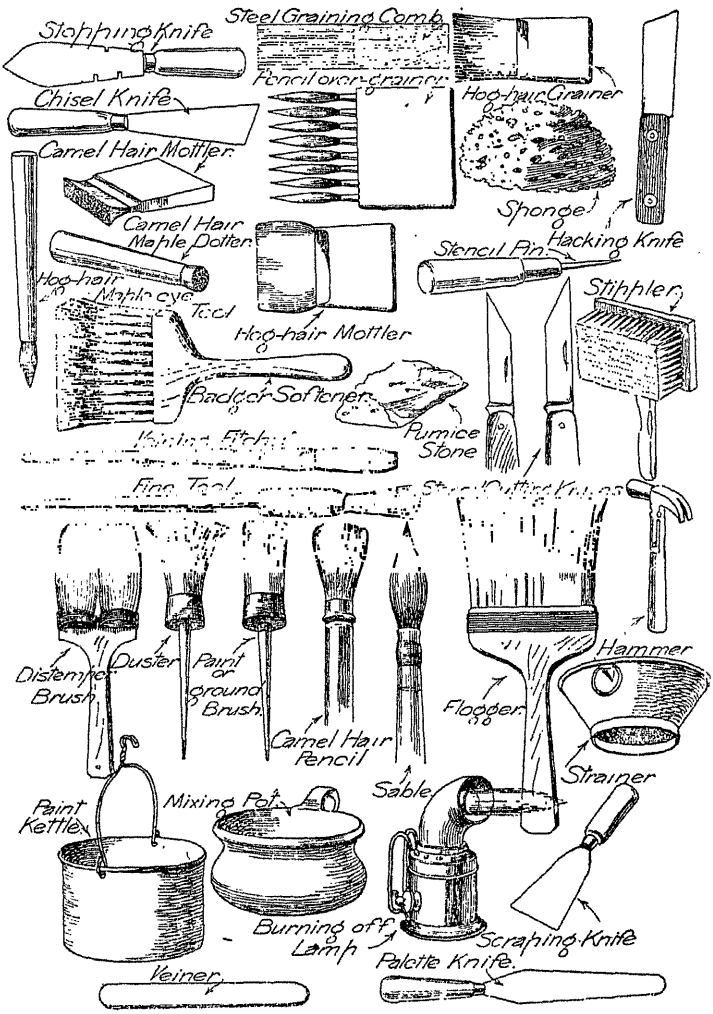
Amber is a transparent yellow substance found in Prussia and on the coasts of the Baltic; it is a bituminous substance, dry, brittle, inflammable, hard, durable and tough, difficult to dissolve, slow in drying, and keeps its colour well, but is very costly. Its inflammability prevents its use in quantities sufficiently large for practical varnish making.

Gum Animé is the name applied to copal, chiefly imported from Zanzibar and Demerara. It is frequently found in rounded masses embedded in sandy soil; it is durable, tough and hard, dries quickly, but is subject to cracking.

Copal, the generic name given originally to all fossil resins, is imported from the West Coast of Africa, South America, Manilla, and New Zealand. It is generally considered the best; is very durable for external work, and is tough and hard.

Driers.—Usually litharge or lead acetate is added to accelerate the drying. Lead acetate is generally considered to be the best, as it combines with, as well as hardens, the varnish. An excessive use of driers injures the varnish and impairs its durability. Good varnish should be quick-drying, hard and tough; should have a good gloss and weather well.

French Polish is a varnish formed by dissolving shellac, a resinous gum, in spirits of wine, and is worked upon the surfaces of hard woods to heighten the effect of the grain. It is applied by rubbing on to the surface of the wood with wadding enclosed in linen rag.



Figs. 59—91.—Painters' Tools.

Wax Polish.—Beeswax in its simple state is rubbed into the pores of the wood, being worked in with rubbers of linen rag, a little turps being added to the rag rubber to make it work more freely. This forms a dull polish on the surface; it is considered far superior, is more durable, and takes longer to accomplish than the French polishing.

Whitewash is made from pure lime mixed with water; it is chiefly used for sanitary purposes; it should be laid on while hot, the operation being known as lime-whiting. It may be coloured by adding earthy pigments, such as ochre, umber, Indian red, and lampblack. It will not stand the weather.

Whiting is made by reducing pure white chalk to a fine powder. Mixed with size and water, it is used for whitening ceilings and walls.

Distemper is the name given to whiting mixed with size and water. It is usually coloured with earthy pigments, such as ochre, umber, Indian red, and lampblack. It comes off if washed, and will not stand if used on surfaces exposed to the weather.

Water Paints.—Of late years many eminent firms have produced a number of proprietary distempers in paste or powder requiring to be mixed with water only. Some of these are claimed to be free from whiting and size. They are made in a great number of pleasing tints, and possess a much greater resistance to washing than ordinary distempers. For interiors these can at times be used with the great advantages of economy, artistic effect, and a moderate resistance to washing, especially in new buildings, where it is often desirable not to decorate the surfaces permanently for 2 or 3 years, or in schools and public buildings, where an annual or frequent application is a sanitary advantage. Duresco, Olsina, Karsonite, Hall's Washable Distemper are well-known examples of these water paints.

Clear Cole.—A size coating applied to fill up the pores of wood or plaster preparatory to distempering or painting.

Painters' Putty is made with whiting reduced to a fine powder and mixed with raw linseed oil.

Painters' Tools.—Figs. 59 to 91 show the common tools of the painter and decorator :—

The mixing pot, palette knife, the strainer, and the paint kettle for the mixing and holding of colour.

The sponge, pumice stone, burning-off lamp, and chisel knife, for the removing of and cleaning down paint.

The scraping or stripping knife used for stripping old paper from walls.

The hammer, hacking, and stopping knives for clearing out broken glass and fixing new.

The stopping knife, two-knot distemper brush, used for clearcoling and distempering ; the duster for removing dust preparatory to applying paint ; the ground brush for broad surfaces ; the tool for cutting in edges ; the fitch and camel-hair pencil for lining and picking out small members ; the sable for gilding ; the flogger for picking up leaf gold ; and a large rectangular brush, with a handle, termed a stippler, for producing a non-streaky rough surface ; the stencil cutting knives and stencil pin.

The following tools are used in graining : the steel graining comb, the hog-hair grainer, the pencil over-grainer, the camel-hair mottler, the camel-hair maple dotter, the hog-hair maple eye tool, the hog-hair mottler, the badger softener, the veining fitch and veiner.

CHAPTER X

GLASS

Glass.—This may be defined as a transparent, non-crystalline solid, consisting essentially of a silicate of an alkali metal combined with calcium or lead silicate.

If a mixture of sand (SiO_2) and caustic soda (NaOH) or caustic potash (KOH) be fused, a silicate of sodium or potassium is formed, which is homogeneous and transparent.

Na_2SiO_3 or K_2SiO_3 is a syrupy solution of sodium silicate or potassium silicate, non-crystalline, transparent, and soluble in water, and is known as water-glass. This is used in the manufacture of artificial stone.

Similarly, with a mixture of sand and lime or of sand and lead oxide, a transparent liquid is formed on fusion. When these silicates are cooled they lose most of their transparency and become crystalline, brittle, and are readily acted upon by water.

If the silicate of soda or potash is mixed with the silica of lime or lead oxide and fused, the mass, when it cools, is a hard transparent glass, no longer crystalline, not readily acted upon by water, and which can be shaped at a red heat.

Window glass is made as (1) Crown, (2) Sheet, (3) Plate glass, and all of these are silicates of soda and lime.

The ingredients used in the preparation of crown and sheet glass are as follows :—

Sand	100 lbs.
Sodium Sulphate	40 to 45 ,,
Chalk or Limestone...	40 ,,
Powdered Anthracite Coal	2 ,,
Cullet (broken glass of the same kind) ...	100 ,,

The introduction of a small quantity of arsenic has the effect of rendering the glass colourless.

Bailey gives upon analysis the composition of a number of different forms of glass as follows :—

	Silica.	Potash.	Soda.	Lime and Magnesia.	Lead Oxide.	Alumina and Oxide of iron.
Bottle glass (ordinary)	65·6	2·7	4·9	20·4	—	6·1
Window glass ...	70·7	—	13·3	13·4	—	1·9
Flint ,, ...	50·2	11·2	—	—	38·1	0·5
Fusible ,, ... (for chemical apparatus)	70·5	2·1	17·2	8·7	—	1·0
Infusible glass ... (for combustion tubes)	73·1	11·5	3·1	10·7	—	0·9

An excess of soda imparts fusibility; in like manner lime-toughness potash-infusibility, used for fire-resisting glass. Lead and potash make glass bright and sparkling; used for table glass. Manganese produces whiteness, and colour is obtained by the addition of metallic oxides.

Crown Glass.—The workman takes at the end of a blow-pipe a bulb-shaped mass of about 10 lbs. of glass, in a viscous state. By blowing through the tube the mass is elongated, and by further blowing it afterwards assumes a flat vase-shaped mass with a bullion point. The mass is then transferred upon a cup-shaped piece of metal which encloses the bullion point. It is carried by an iron rod, known as the ponty, and is taken in front of a furnace, where the rod is at first rotated slowly and then more rapidly, the rim of the vase expanding horizontally till at last it falls into and assumes a flat circular plate with a bull's-eye boss at the centre. The manufacture of this glass has practically ceased.

Sheet Glass.—The process of manufacture is as follows: A mass of molten glass of about 10 lbs. is gathered at the end of a tube and blown out to a pear-shaped mass, then by blowing and swinging simultaneously the mass lengthens and assumes a sack-shaped hollow cylindrical form, the air in the interior is rarefied, resulting in an end collapsing, the edges are then flashed over, after which the upper portion is cut off by a hot string of glass, and the cylinder which remains is cut by a diamond. The cut cylinder is then

placed on a flattening kiln, when gravity in this state causes it to fall flat, after which it is taken to a furnace and annealed. Sheets to the dimension of 10′ × 4′ have been made by this method. The usual sizes of sheets are from 12 to 17 feet in area, and the weight from 15 to 42 ozs. per superficial foot.

The largest sizes that are usually made in the various substances of sheet glass are as follows, but the extreme limits of length and width cannot be combined in the same sheet :—

	Extreme length.	Extreme width.	Extreme area.
15 oz.	60 inch.	40 inch.	15 feet.
21 ,,	90 ,,	50 ,,	26 ,,
26 ,,	90 ,,	50 ,,	25 ,,
32 ,,	85 ,,	48 ,,	21 ,,
36 ,,	70 ,,	44 ,,	17 ,,
42 ,,	70 ,,	44 ,,	15 ,,

The extreme area taken in connection with the extreme length or width required in any particular case will indicate approximately the corresponding limit of width or length.

Patent Plate is the name given to sheet glass by the cylinder process ground and polished. It is higher in price than rolled plate. There are two varieties—the usual, which is preferable for glazing sashes, and the extra white, the lightness and purity of which are valuable qualities for purposes of glazing picture frames and for photographic negatives. It may be obtained of the following thicknesses : $\frac{1}{16}$, $\frac{1}{12}$, $\frac{1}{10}$ and $\frac{1}{9}$ inch ; and up to 50 inches long or 39 inches wide, and 13 feet in area.

Plate or British Plate Glass.—The processes are as follows : 1. Casting ; 2. Grinding ; 3. Smoothing ; 4. Polishing.

1. Rectangular fillets to the thickness of the plate about to be cast are placed about the edges of a smooth iron table ; the molten glass is then poured upon the table, and a roller being worked upon the fillets reduces the mass to the level of the fillets.

2. Grinding consists in placing a rubbing plate of cast iron upon the cast glass, and both glass and plate are made

to move, water is introduced between, and later, powdered emery.

3. After which the glass plates are smoothed, which consists in placing two glass plates and rubbing each against the other with fine emery between.

4. Polishing is done by machinery, the rubbers having epicycloidal motion imparted to them.

The constituents of plate glass may be as follows :—

Fine Sand	100 lbs.
Refined Sulphate of Soda	42 ,,
Carbon in powder	2½ ,,
Carbonate of Lime	20 to 25 ,,
Arsenic	8 ozs.

Cullet (broken glass of the same kind) as may be desired

Sheets may be cast from $\frac{3}{16}$ to 1 inch in thickness, and up to 100 feet in area.

Rough Plate.—If the plate glass is not polished it is known as rough plate.

Rough Rolled Plate.—Plates are cast with a series of fine grooves or flutes, varying from 4 to 11 to the inch, as in Hartley's rolled plate, or patterns may be worked upon the table. This will give corresponding impressions upon the cast plate.

Classification of Glass.—The following gives a classification of glass in common use.

1. Crown Glass (Newcastle Glass).
2. Sheet Glass—(*a*) Plain.
   (*b*) Fluted Sheet.
   (*c*) Dappled Sheet.
   (*d*) Corrugated Sheet.
3. Patent Plate.
4. Plate or Cast Glass—(*a*) Rough Cast Plate.
   (*b*) Rolled Plate : Plain, Small Fluted, Large Fluted, Diamond, Small Quarry, Large Quarry.
   (*c*) Chequered Plate.
   (*d*) Wired Rolled.
   (*e*) Rolled Cathedral.
   (*f*) Polished or British Plate.

Coloured Glass—(*a*) Flashed.
(*b*) Pot Metals.
(*c*) Cathedral Tints.
(*d*) Stained Glass.

WEIGHTS OF VARIOUS MATERIALS.

Materials.	Weight of a cubic foot in lbs.
Water, Pure at 39·4°	62·425
,, Sea, ordinary	64·05
Clay	120
Sand	96·35
Gravel	112·6
Chalk	117 — 174
Shale	162
Marl	100 — 119
Slate	175 — 181
Limestone	169 — 175
Sandstone	144
Granite	164 — 172
Gypsum	143·6
Zinc, sheet	448·1
,, cast	427·6
Tin	454·4
Copper, sheet	547·5
,, cast	536·4
Bronze	524
Brass	523·1
Lead	707·5
Cast Iron	450
Wrought Iron	480
Steel	490
Fir	30
Pitch Pine	39·43
Oak, European	43 — 62
,, American	54
Elm	36·65
Mahogany, Honduras	34·9
,, Cuba	53
Teak, Indian	41 — 55
,, African	61
Glass, Crown	156
,, Flint	187
,, Plate	169
Brickwork Stock, in lime mortar	100 — 110
,, ,, cement mortar	102 — 112
Portland Cement, loose	90

Six and a quarter gallons of water = 1 cubic foot at normal temperature and pressure.

CHAPTER XI

FOUNDATIONS

Definition.—The extended bases of walls, piers, columns, etc., directly supported or kept in equilibrium by the earth, are known as the foundations.

Necessity for Foundations.—Walls of buildings resting on ground of variable strength often fracture, due to the unequal settlement of the work. To prevent failure in this manner, the base of the walls of the building is usually extended and supported by suitable foundations.

The object of foundations is to distribute the weight of the structure equally over the substratum and to prevent inequality of settlement.

The bases of structures are invariably made wider than the superincumbent mass, to increase the stability by distributing the load over an area sufficiently large to safely withstand the pressure and to counteract all the following damaging forces that tend to cause failure.

Damaging Forces.—The principal causes of failure are those which induce settlement, such as inequalities of earth resistance; lateral escape of soft soil; sliding of the substratum on sloping ground; shrinkage due to the withdrawal of water; atmospheric action; distributed lateral pressures, causing overturn, such as wind pressure, and thrust of barrel vaulting or of an untied couple raftered roof; concentrated lateral pressure which induces settlement and overturn, such as the thrust of framed floors, trussed roofs and groined vaults subjecting small areas of support to great pressures.

Inequality of Settlement.—Inequality of settlement in foundations takes place from two causes—(1) the unequal

resistance of the soil; (2) the unequal loading of the substratum.

Nearly all soils, with the exception of solid rock and gravel, are compressible under pressures often attained in buildings. It is therefore impossible, where large buildings are erected on other soils, to avoid settlement; and the fact of any building settling is of no great import, provided the settlement be uniform and of no great depth, and the relative position of the parts of the structure unaltered. But where the resistance of the soil of every part of the site is not uniform, there is a risk of irregular settlement occurring, and secondly, buildings with irregular masses erected on uniformly yielding soils are subject to unequal settlement, due to the varying superimposed pressures. Special precautions must be taken in both cases to distribute the pressure over a sufficient bearing area or by piling.

Lateral Escape.—Soft soils, such as running sands and peat, upon which heavy structures have been erected are liable to squeeze out from beneath the foundation, unless means are taken to confine the soil to the required area; this is usually accomplished by sheet piling, as described later.

Sliding.—This is a defect usually occurring where the building is erected on the slope of a hill, and the strata inclined, being depressed in the direction and towards the bottom of the slope. The weight of the building is liable to cause the strata to become detached and to slide. This is prevented in two ways—(1) by driving piles at intervals to a considerable depth, thus connecting the strata: this method is often objectionable, tending as it does to shake and disturb the soil; (2) by building a retaining wall; this is the better method, as it not only supports, but also protects the strata from the effects of the atmosphere, which in soils easily affected by the latter is a desideratum.

Withdrawal of Water from Foundation Earth.—Edifices built on damp soil, such as a sand overlying a clay, have their stability endangered should the water be drained away after the building has been erected. This action will

cause the soil below the foundations to shrink and tend to produce inequality of settlement; therefore the depth of the concrete foundation must be arranged below the level of any probable adjacent cutting, or a raft foundation be constructed.

Atmospheric Action.—Many otherwise thoroughly reliable soils are practically reduced to the condition of mud if exposed to the effects of the atmosphere and rain-water. The variation in the dampness of the soil at the different seasons also causes the ground to expand and contract considerably.

Where foundations are constructed in such soils, they must be taken sufficiently deep to be beyond the effects of the atmosphere, that is below the line of saturation. Four feet below the ground level is usually sufficient for this purpose, the soil below this not being affected to any appreciable extent by the percolation and subsequent freezing of rain-water.

The line of saturation in the section of any part of the earth's crust represents the depth to which the soil at that part is saturated by the absorption of rain-water, and affected by atmospheric changes. This depth can be ascertained on any site by excavating bore holes or inspection pits.

Distributed Overturning Pressures.—Distributed forces acting upon the upper level of walls, such as the continuous pressure of barrel vaulting and the spreading tendencies of untied couple raftered roofs, and also the distributed pressures on wall faces, such as wind pressure, tend to cause failure in two ways—(1) by overturning, the minimum resistance being generally at any change of section, usually at the ground level; (2) by subjecting the leeward edge of the wall to the pressure sufficient to crush the material or by throwing the weight on a small area of the substratum, forcing it from its original position and causing a settlement.

The stability of walls when subjected to such distributed overturning pressures is treated in the chapter on that subject.

The foundation should therefore be extended on the leeward side of the wall to bring the resultant thrust within the middle third of the width of the foundation.

Concentrated Lateral Pressure.—The thrust caused by untied principals or groined vaults or other forces acting at a point or along vertical lines on the wall are often resisted by buttresses, the foundations of which are constructed as in the previous case.

Soils.—The construction of foundations varies with the nature and bearing strength of the soil. The following are the ordinary soils met with in practice and the method of treating them : Rock, chalk, gravel, clay and sand.

Rock.—Foundations laid upon the solid rock or compact conglomerates of mineral substances are undoubtedly secure, as far as settlement is concerned, such a substratum being practically incompressible. Rocks often have fissures and defective parts ; all such gaps must be filled up with concrete, any unsound parts being cut away. Rock foundations are very expensive in working, owing to the extra labour involved in cutting ; but where they occur concrete is unnecessary except for the purpose of forming a level surface.

Chalk.—Chalk varies considerably in hardness, being in a dry or well-drained position very hard ; but if subject to much wet it becomes saturated and is thus rendered soft.

The sites for buildings on chalk soils should be drained, and precautions taken to prevent them becoming wet. Where this can be done, the structure can be built upon the chalk direct, after it has been levelled ; but where heavy buildings are erected, or great weights concentrated, concrete should be employed to distribute the pressure.

Gravel.—Gravel is one of the best soils to build upon where lateral movement is not likely to occur ; it is not affected by the action of the atmosphere, and is practically incompressible. Sites with thin layers of gravel resting on clay where water can accumulate in the subsoil are unhealthy and should be avoided.

Clay.—Clay is a good soil to build upon where the foundations are taken deep enough to be beyond the action of the atmosphere, and the site of the building is covered with a slab of cement concrete 6 inches thick as required by the model bye-laws. If on a slope which provides a system of drainage it is not unhealthy. It is only the great clay vales that for health purposes should not be selected. Clay is very subject to expansion and contraction with the variations in temperature and is therefore dangerous to build upon unless protected.

Sand.—Sand is a good material to build upon, if it can be kept dry and confined laterally; if subjected to the effects of running water it is liable to be scoured from about the foundation.

In all the above soils, with the exception of the compact rock, and the chalk when in a good condition, it is usual to form a bed of concrete under all the walls of the building, the area of which is proportioned to the weight to be carried and the bearing strength of the soil.

The following table of safe resistances of earth is given by Newman :—

Description of Earth.	Approximate safe Maximum load in tons per square foot.
Bog, morass, quicksand, peat moss, marsh land, silt	0 to 0·20
Slake and mud, hard peat turf	0 to 0·25
Soft wet pasty or muddy clay, and marsh clay ...	0·25 to 0·33
Alluvial deposits of moderate depths in river beds, etc.	0·20 to 0·35
Note.—When the river bed is rocky and the deposit firm, they may safely support 0·75 tons, but not more.	
Diluvial clay beds of rivers	0·35 to 1·00
Alluvial earth, loams and loamy soil (clay and 40 to 70 per cent. of sand), and clay loams (clay and about 30 per cent. of sand), damp clay... ...	0·75 to 1·5
Loose sand in shifting river bed, the safe load increasing with depth	2·50 to 3·00
Upheaved and intermixed beds of different sound clays	3·00
Silty sand of uniform and firm character in a river bed secure from scour, and at depths below 25 feet	3·5 to 4·00

Description of Earth.	Approximate safe Maximum load in tons per square foot.
Solid clay mixed with very fine sand	4·00
Note.—Equal drainage and conditions are especially necessary in the case of clays, as moisture may reduce them from their greatest to their least bearing capacity. When found equally and thoroughly mixed with sand and gravel, their supporting power is usually increased.	
Sound yellow clay containing only the normal quantity of water	4·00 to 6·00
Solid blue clay, marl and indurated marl, and firm boulder gravel and sand	5·00 to 8·00
Soft chalk, impure and argillaceous	1·00 to 1·50
Hard white chalk	2·50 to 4·00
Ordinary superficial sand beds	2·50 to 4·00
Firm sand in estuaries, bays, etc.	4·50 to 5·00
Note.—The Dutch engineers consider the safe load upon firm clean sand as 5½ tons per square foot.	
Very firm compact sand foundations at a considerable depth not less than 20 feet, and compact sandy gravel	6·00 to 7·00
Note.—The sustaining power of sand increases as it approaches a homogeneous gravelly state.	
Firm shale, protected from the weather in clean gravel	6·00 to 8·00
Compact gravel	7·00 to 9·00
Note.—The relative bearing powers of gravel may be thus described :—	
1. Compact gravel ; 2. Clean gravel ; 3. Sandy gravel ; 4. Clayey or loamy gravel.	
Sound, clean, homogeneous Thames gravel has been weighted with 14 tons per square foot at a depth of only 5 feet below the surface, and presented no indication of failure. This gravel was similar to that of a clean pebbly beach.	
Rocks for foundations and general work	8·00 to 18·00
Rocks, sandstones that may be crumbled in the hand	1·50 to 1·75

Pressures on Subsoil.—From the British Standard Specification No. 449. Permissible loads are given below as a general guide to the safe bearing capacity of various subsoils. Trial holes or loading tests shall be made, or other measures taken which may be necessary to ascertain the safe bearing load of the ground upon which the foundations of a building are to be bedded.

Note.—It is advisable to consult the local building authority.

Intermediate values and values for other subsoils shall be agreed in consultation with the local building authority.

	Permissible load in Ground Tons per sq. ft.
Alluvial soil, made ground, very wet sand ...	Up to ½
Soft Clay, wet or loose sand	,, 1
Ordinary fairly dry clay, fine sand, loam ...	,, 2
Firm dry clay	,, 3
Compact coarse sand, confined sand, London blue clay and similar hard compact coarse gravel ...	,, 4
Hard Solid Chalk	,, 6
Shale and Soft Rock	,, 10
Hard Rock	,, 40

These pressures may be exceeded by an amount equal to the weight of the material in which a foundation is bedded and which is displaced by the foundation itself, measured downward from the final finished lowest adjoining earth level, or the upper level of any solid raft directly on the earth.

Weights of Earth.—The following weights of different earths are given by Newman :—

Name of Earth.	Weight.	
	Decimals of a ton. Cubic foot.	Tons. Cubic yard.
Basalt, solid	0·083	2·25
Bath Stone, solid	0·052	1·40
Chalk, damp to wet, loose to close ...	0·056 to 0·074	1·50 to 2·00
Clay	0·054 to 0·059	1·45 to 1·60
Flint, solid	0·074	2·00
Granite	0·078	2·10
Gravel and Shingle	0·046 to 0·055	1·25 to 1·50
Limestone Lias to Compact Mountain	0·067 to 0·078	1·81 to 2·10
Marl	0·044 to 0·052	1·20 to 1·40
Mud at surface...	0·044	1·20
Mud at about 15 ft. in depth ...	0·048	1·30
Peat, hard and top mould	0·036	0·98
Portland Stone, solid	0·065	1·75
Quartz, solid	0·076	2·05
Sand, dry river	0·041	1·10
Sand, damp and shaken	0·055	1·50
Sandstone, solid	0·063 to 0·072	1·70 to 1·95
Shale	0·074	2·00
Slate, solid	0·08	2·15
Trap, solid	0·078	2·10

A cubic yard of water weighs 0·752 tons.

Sanitation of Soils.—Soils and subsoils may become injurious to health by the presence of dampness or the growth or putrescence of vegetable matter, which give off noxious emanations producing consumption, lung and kidney diseases, neuralgia, rheumatism, etc. Other conditions being equal, low-lying districts are less healthy than those on high ground or the slopes of hills with good natural drainage. Dryness being a most important factor, it is necessary, first, in all soils to have a good system of subsoil drainage at a reasonable depth; secondly, a paved area immediately adjacent to and about all buildings; and thirdly, a layer of impervious concrete over the whole of the site beneath the house, and all parts of the building below the ground level should be made damp resisting.

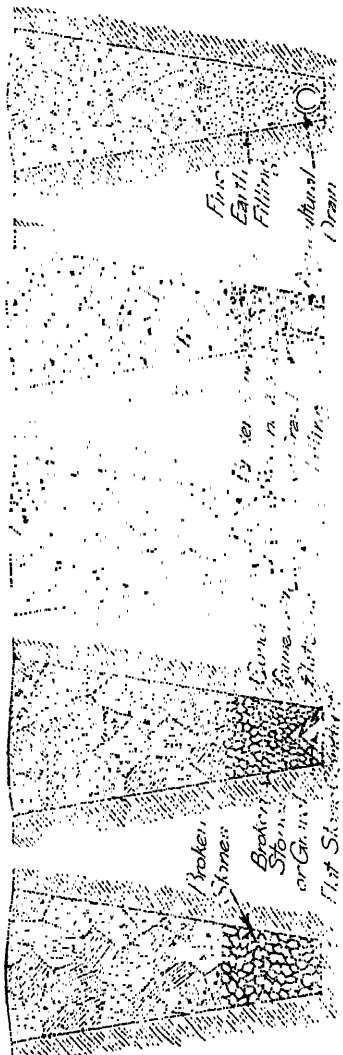
It is impossible at all times to select the site or change the character of the soil, yet it is possible to render all soils healthy by proper application of the above four conditions. The subsoil drainage will lower the permanent level of water and render the surface soil dry, the paving of areas prevents the percolation of water through the soils, and with proper falls the water may be directed to the drainage system and thus prevent the soil becoming wet. The concrete over the site sterilizes the soil, preventing any organic growth under the buildings and the emanation of ground gases, the damp-resisting walls acting for a similar purpose.

Subsoil Drainage.—In low-lying districts, and in damp soils generally, the site for any building should be drained thoroughly before the structure is commenced, especially if there is the remotest possibility of any future cuttings being made, such as would be required for sewers or railways, which, by acting as a drainage system, might cause the failure of the foundations. Dampness in the soil has a deleterious effect upon the health of the inmates. It also causes defects in the building; first, by the expansion and contraction of the earth, consequent upon the absorption and evaporation of moisture, which tends to rend the walls; secondly, the damp is drawn up the walls. The latter may be stopped by an efficient damp-proof course. If the ground floor is constructed with timber the latter will be

subject to wet and dry rot. These may be prevented by proper damp-proof courses, concreting over site, and thorough ventilation. To reduce the possibility of this defect, a layer of concrete at least 6 inches thick should be spread over the whole area covered by the building, and a damp-proof course over the whole site, as shown in Chapter on Brickwork (*Elementary Course*).

Where isolated buildings have to be erected, the method of draining the land would be as follows: A trench or ditch is dug about the whole site to intercept any water that may flow over the land, and prevent it passing over the site. The site is divided into a number of parallel bays by narrower trenches than the above, all having a fall towards the lowest part of the site. Between these a number of still smaller trenches are cut, being arranged to a herring-bone pattern. These diverge from the centre line of the bay in the direction of the fall, and discharge into the above-mentioned trenches; the latter empty themselves into the enclosing ditch, from which the water is conveyed by a continuation of the latter, or by a pipe, to the nearest stream, if the land is nearly level; or if the ground is on the slope, the water is discharged over it at a lower level.

Where an estate has to be laid out it should be carefully surveyed and the levels taken, preferably by contouring, to locate the natural drainage system of the land. The natural drains, *i.e.*, the brooks and streams, always occur at the lowest portions of the particular section of land they drain. These brooks frequently require deepening and the banks require to be properly defined and made up owing to their tendency to become silted and to render the land in the immediate vicinity marshy by overflowing their banks during the wet seasons. The land on either side of the brooks may then be drained by excavating open ditches with proper falls, their direction being normal to the contour lines. The distance between the ditches depends on the porosity of the soil and must be determined in each case. Open ditches are objectionable on residential property, and also, if many are required, on agricultural land. In these cases drain pipes with open joints are laid in the bottoms of the ditches, being surrounded by a porous material such as gravel, and the ditch is filled in,



Figs. 92—96.—Methods of Forming Surface Drains.

preferably with a porous soil. In addition to these main drains, tributary drains laid in a similar manner, at distances varying from 10 to 30 feet apart, are put in, herring-bone in plan. By thus taking advantage of the natural levels of the ground and traversing the contours, large areas otherwise marshy may be economically drained and rendered dry, healthy and constructionally safe.

Trenches.—The depth of the trenches usually varies from 2½ to 6 feet; they are cut as narrow as possible. The bottoms of the trenches are cut to regular falls of not less than 1 in 100. The distance between the smallest trenches varies with the soil, being about 10 feet in clay to 30 feet in lighter soils.

A duct is formed at the bottom of the trenches for the conveyance of water. This may be done in one of

three ways—first, by placing a layer of broken stones for a depth of about 1 foot at the bottom of the trench, and filling in over with the excavated earth, as shown in Fig. 92. Secondly, in districts where a thinly bedded, highly stratified stone is readily obtained, a duct is formed by placing two stones leaning together at their top edges, making a triangular aperture as shown in Fig. 93, or with three stones arranged as in Fig. 94. These stones may be covered with a layer of broken stones or gravel, as shown in Fig. 93, or the earth, if porous, may be filled in direct, as shown in Fig. 94. Thirdly, agricultural drain pipes are laid at the bottom of the trenches and filled in with a layer of broken stone, screened cinder or gravel, as shown in Fig. 95, or with fine earth, as shown in Fig. 96. The pipes are laid dry, with a butt joint in the bottom of the trench; the ends are placed close together, the uneven surfaces leaving a sufficient space for the water to find its way through. Collars are sometimes placed over the joints to prevent earth finding its way in, and to prevent the ends of the pipes getting removed from each other while the trench is being filled. The collars consist of a piece of pipe of a larger diameter than the drain, into which the ends of the latter fit loosely.

Filling In.—The space above the stones, tiles, or pipes should be filled in with a fine porous earth to a depth of at least 15 inches, and above this the ordinary earth may be placed, the latter being shot in lightly at first, but finally well rammed.

Failure and Prevention.—Such a drainage system is liable to failure from the following causes: (1) By the accumulation of silt and vermin in the pipes; (2) roots of trees, which push the pipes out of their place and extend up the pipe, and finally stop them up; (3) where the pipes are under a building they are liable to fail by the settlement of the latter.

(1) The first danger may be avoided by building a catch-pit at intervals, which consists of a brick chamber, into which the pipe discharges; all the matter in suspension falls to the bottom of the chamber, the fluid flows off

through the continuation of the pipe on the opposite side of the catchpit. To prevent vermin, such as mice, etc., from getting up the pipes, all the outlets should be covered by a wire guard or broken glass.

(2) If laid near trees, collars should be employed, or socketed pipes set in cement.

(3) If occurring under the walls of a building, a space should be left and arched above them, in order that the pressure may not be brought to bear on the pipes on the settlement of the building and consequent compression of the earth.

In the Model Bye Laws issued by the Ministry of Health, the following requirement is made: "The subsoil of the site of a building (other than a building of the warehouse class intended to be used wholly or principally for storage or the accommodation of plant) shall, wherever the dampness or position of the site renders the precaution necessary, be effectually drained, or such steps shall be taken by the construction of a layer of impervious materials upon the site as will effectually protect the building from damp arising from the subsoil." No pipe shall be laid in such a manner or in such a position, as to communicate directly with any sewer or cesspool, or with any drain constructed or adapted to be used for conveying sewage, but a suitable trap shall be provided with a ventilating opening at a point in the line of the subsoil drain as near as may be practicable to such trap.

Preliminaries to Building.—The following conditions should be complied with: (1) The requirements of the building owner; (2) the requirements of the local authorities; (3) the rights of adjoining owners must not be infringed; (4) plans and working drawings for the execution of the work.

First, the site must be surveyed and the levels taken, and the exact position of site as outlined on the deeds must be confirmed, to avoid subsequent disputes with adjoining owners of land, as it is a frequent cause of litigation in urban districts where the sites are so often covered to the limits of their boundaries. In London, where the properties are separated by a common party wall, definite regulations govern the rights of adjoining owners, and

notices must be served when pulling down and re-erecting a building which has a common party wall or when a building is erected within 20 feet of adjoining property. For details see the relevant section of the current London Building Act. Plans of the proposed building are drawn up to meet the requirements of the building owner. Drawings of all proposed work must be submitted to the local authorities for their approval according to their published requirements. When the above are satisfactorily settled, contract drawings for the proposed works are prepared. If of a justifiable magnitude, and the work is to be competed for, a bill of quantities should be prepared, copies of which are supplied to the various contractors as a basis for tendering; all other conditions being equal, the lowest should obtain the contract if the work is carried out. For the various conditions affecting a building contract, reference should be made to the form of building contracts issued by the Royal Institute of British Architects, which document contains a series of clauses usually required to be agreed upon. If not relevant to the particular work, any of these may be deleted.

Levelling and Setting Out.—Before commencing any constructional work in connection with a building it is necessary as the first operation to take carefully the levels of the site, in order first to arrive at an estimate of the amount of earth work to be done; and secondly, to determine the design of the basement storey, this latter often being materially affected if the differences in level of the various parts of the site are great. The next operation is to level the ground. This in most instances consists in excavating and removing parts of the site and depositing in other parts to form embankments, or to fill up hollow places. In order to conduct these operations in the most economical manner the levels must in all instances be taken and plotted with the greatest accuracy. This can only be efficiently done on areas of any magnitude by means of the surveyor's level, the method of employing which will be described later. All levelling operations for ordinary constructional work may be carried out by referring them to the principles laid down for performing the following three operations :—

1. Taking levels of site.
2. Levelling the bottoms of trenches for drains or foundations.
3. Embanking for roads or levelling of depressions.

Instruments.—The instruments required to determine the levels of the site are—first, the surveyor's level; secondly, the measuring staff; thirdly, ranging poles; and chain or tape.

Levelling.—The first operation is to fix a datum level, to which all other levels are referred. A line carved on the walls of some permanent structure is preferred, if such does not exist within a convenient distance of the work, a stone set in concrete or if the magnitude of the work does not justify this expense, a stout stake is driven in the ground. Both the latter should be in some position on the site where they are not likely to be disturbed by the building operations. The datum for important works is usually referred to the Ordnance Datum.

For small sites where the surface is fairly uniform, it is only necessary to take the levels of the boundaries of the site. The altitude of any point on the area of the site can be determined by drawing a line through the point to intersect two of the boundary lines, and setting up a section of this line. A second method is to set up a series of parallel sections across the site and marking on the plan the heights of all the points recorded. These are known as spot levels.

The process of taking the levels of a section are as follows. The line is first ranged out with ranging poles. The position of the first point is located with reference to some known point and its bearing is taken or its final point is located on the boundary it intersects. The level is then set up either on or in close proximity to the line. The measuring staff is then held alternatively on the datum and the first point on the extremity of the line, and their relative heights are recorded in the field book. A number of points on the line are then taken, and the measuring staff is held over them, and their relative heights are recorded, and their distances from the beginning of the line are measured. When the bottom of the measuring

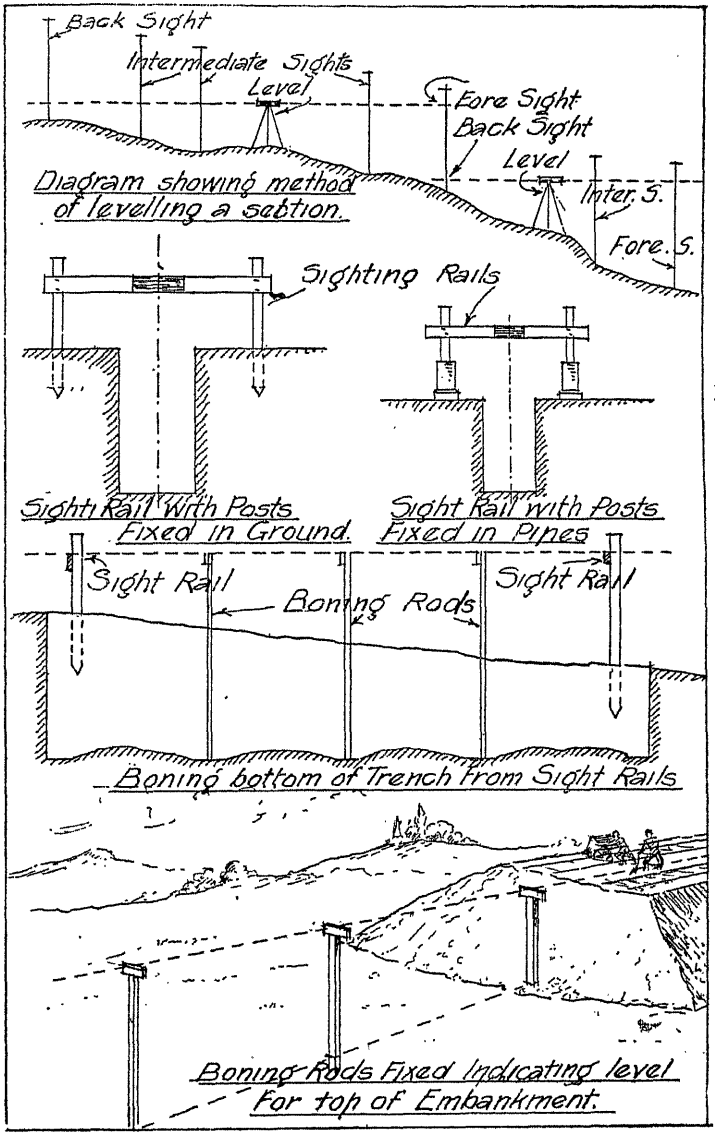


Fig. 97.

Fig. 98.

Fig. 99.

Fig. 100.

Fig. 101.

Figs. 97—101.

staff rises above, or its top becomes depressed below the line of sight, through the rise or depression of the ground, the level must be moved further along the line and the preceding operations repeated. Fig. 97 illustrates the method. The following is a form of field book with the readings for a section entered :—

FIELD LEVEL BOOK.

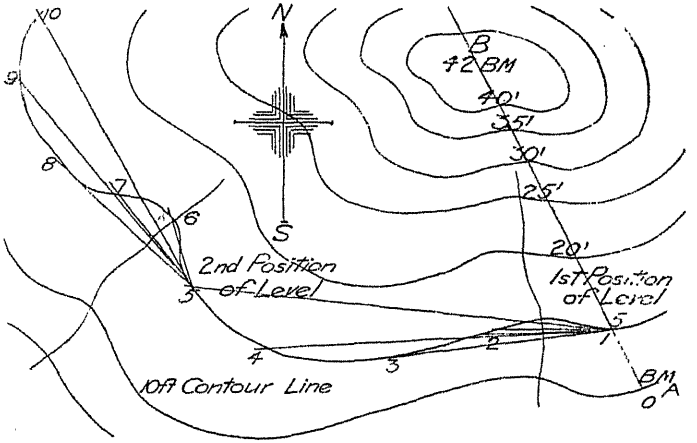
Back Sight.	Inter. Sight.	Fore-Sight.	Rise.	Fall.	Reduced Levels.	Distance.	Total Distance.	Remarks.
						chains		
4·15	...	...	...	...	100·0	...	...	Bench Mark A
	4·13	...	·02	...	100·02	1	...	1 peg
	5·01	...	...	·88	99·14	2	...	2
	4·86	...	·15	...	99·29	3	...	3
	6·06	...	...	1·20	98·09	4	...	4
		8·02	...	1·96	96·13	5	...	5
12·25								
	8·46	...	3·79	...	99·92	6	...	6
	3·04	...	5·42	...	105·34	7	...	7
		2·15	·89	...	106·23	7·57	...	Bench Mark B
12·60								
	7·19	...	5·41	...	111·64	8·57	...	8
		2·53	4·66	...	116·30	9·57	...	9
9·37								
	5·75	...	3·62	...	119·92	10·57	...	10
		3·94	1·81	...	121·73	11·57	11·57	Bench Mark C
			25·77	4·04	21·73			
			4·04					
			21·73					

The above shows a typical field level book. The reduced level of the first point is taken 100 feet above a datum level; the levels are all read in feet and hundredths of a foot; the distances are taken in chains and links, but may be taken in feet and inches. The rise and fall columns should be balanced, also the first and last reading in the reduced levels; these two quantities will equal each other if the computations have been correctly made.

Contouring is the process of delineating a series of level lines on a map or plan of uniform difference of level; this

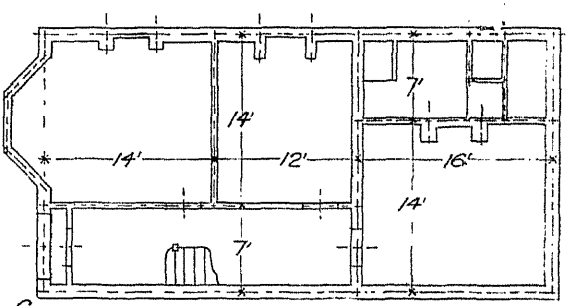
difference is known as the vertical interval. The distances between the lines measured on the map or plan are the horizontal distances, and usually they are irregular. This readily affords a means of representing on a plan to a trained mind the irregularities of the earth's surface.

The method of contouring is the most useful, but it takes the longest time to carry out. To perform this operation it is necessary to fix a bench mark as a starting point. This might be convenient at the lowest boundary of the site, as illustrated in Fig. 102, but it is easier in working to start from the topmost point. At contour 10 feet level it would be well to determine the relative height of this bench mark with reference to the Ordnance levels, but this is not always necessary. A line is then ranged from the bench mark to the summit or highest point on the site. The bearing of the line is recorded and the required vertical intervals are located with the surveyor's level, a stake being driven in at each point as shown on the line A—B, Fig. 102. These stakes are numbered, and then are made the starting point for any contour line, the process being as follows: The level is erected over the numbered peg; the height to the centre of the telescope is taken with the staff; the assistant with the measuring staff then walks some convenient distance, such as thirty paces; the surveyor operating at the level, directs the assistant up or down until the height recorded on the measuring staff is identical with the height of the level of the axis of the instrument or line of collimation. The bearing of the line and the distance of the measuring staff from the instrument is then recorded. The distance may be measured with the chain or by the subtense points or lines, usually fitted to modern instruments. The latter method is to be preferred, as it is far more rapid and sufficiently accurate for the purpose. This process is repeated to as many points on the contour at intervals of about thirty paces apart, as can be observed from the instrument. The level must then be removed and set up at the last observed point, and the process repeated until the boundary is reached or the circuit is completed. The following is the form of field book employed, and Fig. 102 illustrates a contoured plan with the setting out lines :—



Methods of Contouring

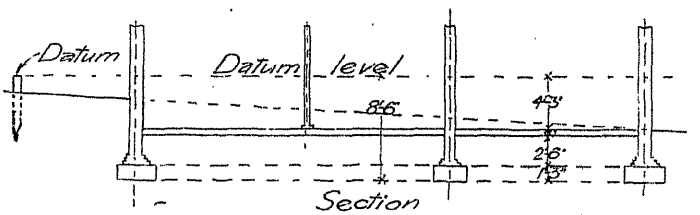
Fig. 102.



Plan of ▣ Datum

Plan

Fig. 103.



Method of Setting out Foundations

Fig. 104.

Figs. 102—104.

CONTOUR BOOK.

Number of Contour.	Height of Instrument.	Bearing.	Distance.	Remarks.
15′ — 0 Contour	4′ — 0	270° — 10′	35 — 0	
		263° — 15′	68 — 0	
		267° — 34′	109 — 0	
		275° — 15′	130 — 0	
	5′ — 0	343° — 5′	21 — 0	
		321° — 20′	36 — 0	
		312° — 40′	55 — 0	
		319° — 30′	83 — 0	
		330° — 15′	96 — 0	

Another method and much more rapid, if convenient, is to start from the topmost point and set out a sufficient number of lines to the boundary of estate with angles as 20, 40, 60 degrees, etc., and on these with surveyor's level and measuring staff determine the contour points 5′, 10′, 15′, etc., planting numbered stakes to indicate the points. All the necessary and salient points on the 5′ and 10′ circuits can usually be taken without moving the levelling instrument from the top. The remaining part of the contour levelling may easily be inferred from the description of the previous method.

The frontage line of the face of the building is first determined. In urban districts this usually has to conform to the building line determined for the particular street by the local authority. In the case of new streets the building line is usually a prescribed distance from the centre of the road. In the case of rebuilding the frontage may be determined from existing buildings on either side. This frontage line constitutes the datum vertical plane to which the position and direction of all other walls are referable. In the case of detached buildings on an open site the problem is easier. When once the frontage of the building has been located and its limits pegged out,

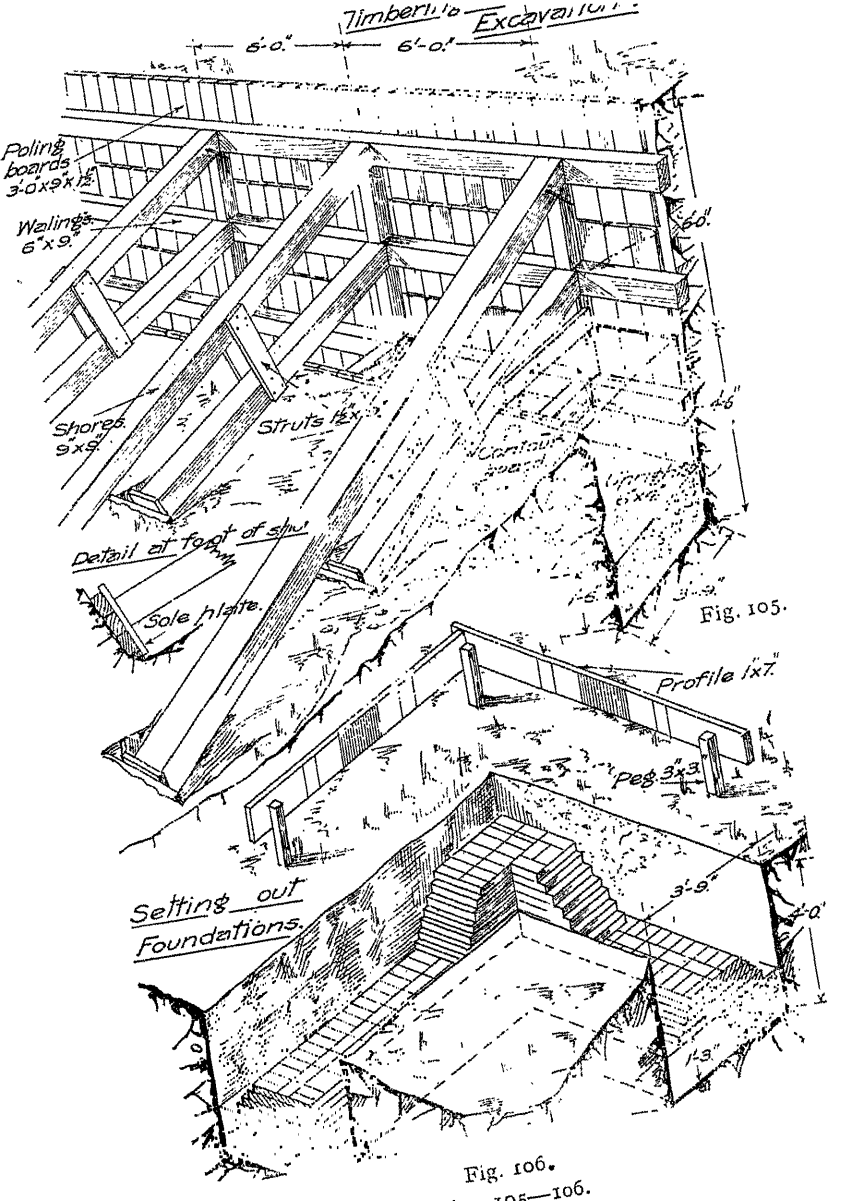


Fig. 105.

Fig. 106.

Figs. 105—106.

profiles are fixed usually a few feet back from the angles of the walls, as shown in Fig. 106. These consist of pieces of wood about 6 inches in width securely nailed to stout pegs driven well into the ground and, if necessary, backed up with concrete. On these are marked the centre lines of the walls, the thickness of the walls, and the extension of the footings and concrete on either side of the centre line. The top of the profile is employed as a sight rail and is fixed either at datum level or at a given distance

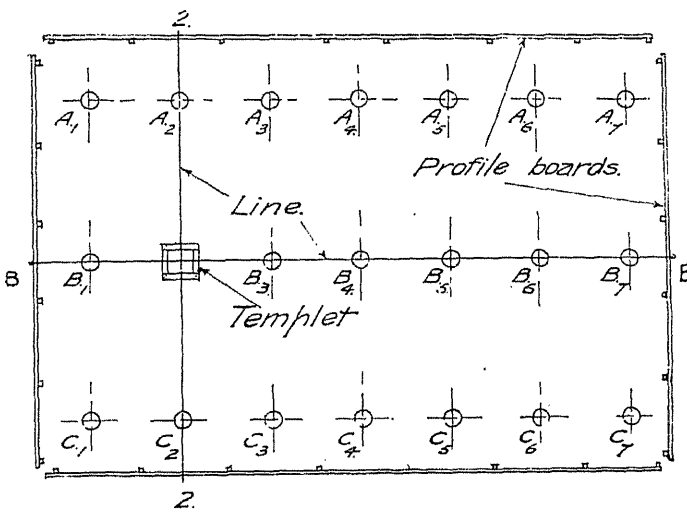


Fig. 107.

above or below it. They are then used with the aid of a boning rod in determining the depth of the trench, the level of the concrete, etc. In fronts having a considerable number of breaks or projections the profiles are usually made continuous about the walls, and on them the exact position of each point is accurately located. The position for projecting bays, buttresses, breasts, etc., are then determined with the aid of templets of the form required, laid with reference to the face of the wall of which they form a part.

In the case of steel frame buildings where accuracy is

essential and any failure in this respect will cause endless trouble and expense throughout the whole height of the structure, it is necessary to have profiles about the whole of the building, as shown in Fig. 107, in order that the centrelines of the stancheons can be determined by the intersection of two lines strained between the points on the profiles. The position of the foundation excavation being exactly located by a templet, as shown in Fig. 107, set in position with reference to the lines. Steel-framed structures are usually set out and the lengths cut and the connections are made in the structural engineers' workshops, the lengths of the joists, etc., being cut exact up to a sixteenth of an inch, and as any alterations can only be made at great expense, it is obvious that all precautions should be taken in the setting out to have all the bases or foundation grillages accurately located. As an extra precaution, it is usual to bolt the horizontal beams throughout at least two floor levels before grouting the bases of grillages.

For measuring distances in setting out, steel tapes or wood rods should be employed, preferably the latter, as there is always a tendency to gain in the former due to the sag in the tape. For levelling pegs, etc., the dumpy level should be employed as more exact work can be realized with this than by any other method. For setting out cross and return walls, large wood squares or templets are commonly employed, though this work can always be more satisfactorily done with the theodolite, especially in the case of walls making angles other than ninety degrees with the frontage line. Walls built to a curve of large radius have the curve pegged out, either by erecting ordinates from a chord line or by means of a theodolite, The pegs should be driven at uniform intervals, usually at the centres of piers. At these points profiles are placed above and normal to the proposed curve; the section of the trench between two profiles can then be excavated or the wall built to a templet of the required curve placed between the profiles.

Boning Method of Levelling.—This operation is used for the levelling of trenches, ground work, paving, etc. There

are three rods in a set, two of these are levelled at a distance of about 10 feet apart: a third rod is then levelled at a similar distance, taking care to reverse the long level. The centre rod is then removed, and the level transmitted to any sight along the line by sighting or boning over the first and third rods. The method of using boning rods and setting a kerb-stone is shown in Chapter on Brickwork (*Elementary Course*).

Trenching.—When the lines of the building have been laid down and all its salient angles pegged out, the work of excavating the trenches commences. It is absolutely necessary that the trenches should be level along their bottoms. To ensure this two or more sight rails (as shown in Figs. 98 and 99) are erected over the trench; it is necessary that the side posts of these should be fixed in such a position that they shall not be disturbed by any of the subsequent operations. A level line is sighted through the level and marked on the sight rails; the cross bar is then fixed on each, and a mark is made on the bars plumb over the centre of the trench. The width of the trench is marked out with the line and pegs, and the excavation is carried on, timbering being inserted as the earth is removed if required, by one of the methods afterwards described. When the full depth of the trench has been nearly reached, a number of points are sunk to the exact depth by means of boning rods, the top of which is sighted between two of the sight rails, as shown in Fig. 100. The remaining parts of the trench bottom are then taken out level between the points so determined. A similar process is employed for sinking a trench for a drain, the variation being that the sight rails have a difference in height necessary to give the required fall.

Embanking.—The method of forming an embankment is as follows: The centre line of the proposed work is ranged out on the ground, and at equal intervals along the line boning rods are erected, the two extreme rods being first fixed either level or with a difference in height sufficient to give the required gradient; a rod is then erected on each of the intervals determined upon, and boned

between the two extreme rods. The embankment is then commenced from one end, the earth being tipped in from carts or waggons until the tops of the boning rods are reached. Sufficient earth in excess must be allowed to compensate for compression and settlement. The width of the embankment is completed as the work is pushed forward, as shown in Fig. 101.

Timbering for Excavations.—The cutting of trenches requires to be carried out with considerable care, particularly if the trenches are to be left open for any length of time, as there is a danger of the moisture draining or drying out and the sides of the excavation falling in. As a rule with firm earth, the ground may be sunk to a depth of from 4 to 6 feet without any support to the sides, if the cutting can be filled in reasonably quickly. But above six feet in depth any earth should be timbered as vibration or the withdrawal of water will inevitably cause the sides to collapse. For shallow trenches in firm ground, open timbering as shown in Fig. 108 can be employed. This consists of pairs of poling boards, 3 to 4 feet in length placed at distances of 6 feet apart and fixed by struts. In ground that is less firm the second method of open timbering shown in Fig. 109 is used, here the poling boards which are usually $3' \times 9'' \times 1\frac{1}{2}''$ are placed along the sides of the trench a distance of about 9 inches apart, walings, horizontal timbers from 6 to 9 inches wide by about 4 inches thick are placed in pairs against the poling boards on each side of the trench and are strutted apart by stout struts. The walings should be placed in the centre of the polings. In cuttings above 6 feet in depth, or in loose soil, the sides should be close boarded, in this case the polings are placed close together and waled and strutted as before. It is essential to prevent the escape of earth from between the boards, this is very liable to take place after heavy rains, and any lessening of the resistance behind the boards will cause the timbering to collapse with very little warning, with consequent danger to the operatives at the bottom of the trench. Where the trenches are liable to remain open for any length of time, and with deep trenches this must always be the case,

the walings and struts must be of ample dimensions as the pressures will be considerable. A general rule for the approximate intensity of pressure at any depth, may be derived from Rankine's formula :

$p = 40h$ where p = pressure per square foot in lbs.
h = depth of cutting.

Owing to the cohesive value of most earths the full earth pressure does not act for a considerable time. The above value may be safely reduced in most cases to $p = 20h$, but the excavators must be guided by observation of the condition of the soil, which may in some cases require the full value of $40h$ to be taken. Thus for a cutting say 5 feet wide by 20 feet deep, the struts being placed at 6 feet centres. The pressure on the waling equals

$p = 20h = 20 \times 20 = 400$ lbs. per square foot
and $P = 6 \times 3 \times 400 = 7200$ lbs.
$= 3{\cdot}25$ tons.

Then taking the depth of the timber as 9 inches obtain the thickness as follows :—

$$\frac{Wl}{8} = \frac{fdt^2}{6} = \frac{3{\cdot}25 \times 72}{8} = \frac{{\cdot}75 \times 9 \times t^2}{6}$$

$$t = \sqrt{\frac{3{\cdot}25 \times 72 \times 6}{8 \times {\cdot}75 \times 9}}$$

$t = 5{\cdot}1''$, say $5''$.

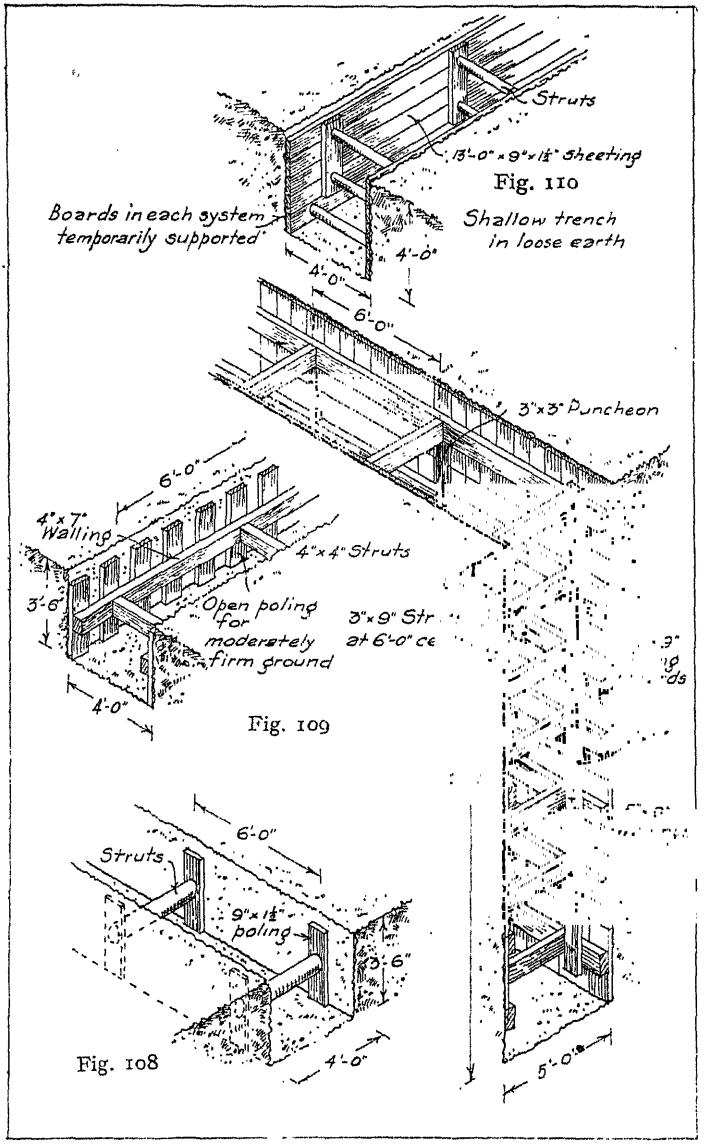
Let the thickness of the strut be 3 inches if the length is 5 feet. From graph in *Elementary Book*, Fig. 585.

$P = 3{\cdot}25$ tons $\frac{l}{d} = \frac{60}{3} = 20$ and $p = {\cdot}44$ tons per square inch.

Then $\frac{P}{p} = \frac{3{\cdot}25}{{\cdot}44} = 7{\cdot}4''$, say $3'' \times 3''$.

This would not afford a proper support for the walings, use $9'' \times 3''$.

Where a series of heavy walings in a deep trench occur one under the other, it is usual to support the upper walings by short puncheons resting on the lower walings continuing these to the bottom, and finally the whole series being connected by a continuous plank spiked to the walings at intervals. These two precautions are taken to prevent the timbering collapsing suddenly if


Struts
13'-0" × 9" × 1½" Sheeting
Fig. 110
Boards in each system temporarily supported
Shallow trench in loose earth
4'-0"
4'-0"
6'-0"
3"×3" Puncheon
6'-0"
4"×7" Walling
4"×4" Struts
3'-6"
Open poling for moderately firm ground
3"×9" Str
at 6'-0" ce
4'-0"
Fig. 109
6'-0"
Struts
9"×1½" poling
3'-6"
4'-0"
Fig. 108
5'-0"

Figs. 108—111.

there is any loosening of the struts, due to the shrinkage of the timber or earth on drying, or to its escape through gaps in the poling or up from the bottom. See Fig. 111.

Shallow trenches in very soft ground are sometimes sheeted. That is the sides are lined with ordinary scaffold

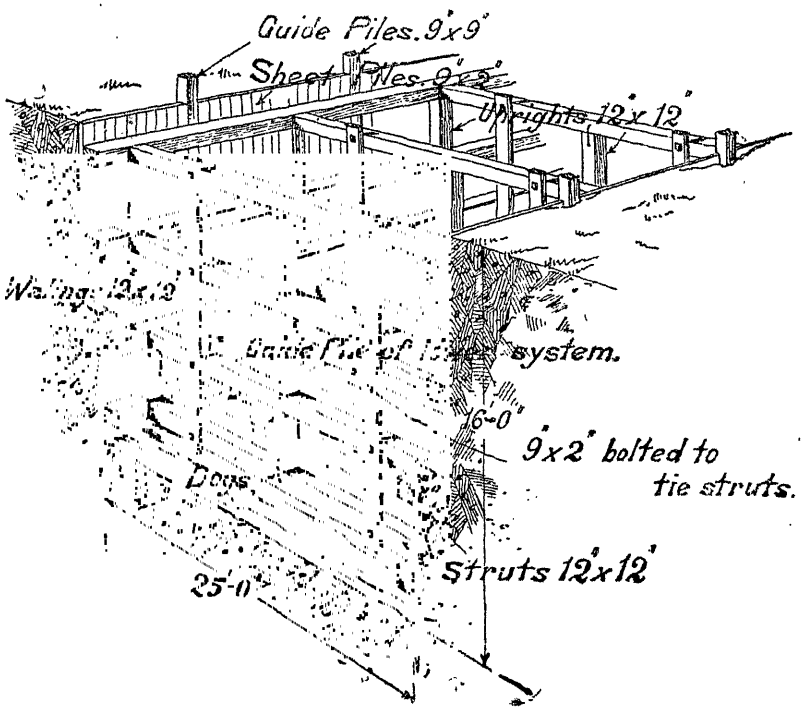


Fig. 112.

boards 13′ × 9″ × 1½″. The ground is excavated 9 inches depth at a time, the sides lined with a pair of boards, these being temporarily strutted. When the full depth is attained, vertical poling boards are placed in pairs on each side of the trench against the sheeting and strutted. The temporary struts are then removed, see Fig. 110.

When cuttings are required over 4 feet in depth in such soils, the methods of runners should be employed, see p. 255.

Fig. 105 shows the method of timbering employed where the ground has to be excavated for a basement and the foundation trench sunk for the retaining wall to support the earth outside the building. In this case the ground is excavated for the basement, and for the first section about 3 feet in depth of the timbering. The dotted lines in Fig. 105 show the earth left in at this stage. The top row of poling boards and walings and the top system of shores are then fixed. The earth indicated by the dotted line is then removed and the next system of timbering and shores is fixed. The trench for the foundations is excavated and timbered. Profiles can then be nailed to the upper timbering for the levels of the trench and the position of the walls.

Large Cuttings.—Continuous trenches, if made in bad ground, are generally arranged as shown in Fig. 112.

At intervals guide piles are driven in, to which walings are bolted, and sheeting consisting of boards about 10 feet long, shod with iron, termed runners, inserted between; these are driven a short distance into the ground, the earth between the two systems of piles being then taken out, and care taken not to excavate within a foot of the bottom end of the runners, which are again driven in and the process repeated. After the excavation of the first part, wales, consisting of whole timbers, are placed in position and strutted apart, the struts being also of balk timber. Long struts are supported in the direction of their length by short uprights secured to them by dogs. Uprights are also placed between the waling pieces as each fresh one is inserted.

After the ground has been excavated to the depth of the runners, a fresh system of piles and runners is driven slightly in advance of the former system, and the ground excavated as before. Cuttings are made in firm ground by excavating the earth and using ordinary sheeting, but if the cuttings are required to exceed 30 feet in width, it is found to be more economical to adopt a system of raking shores.

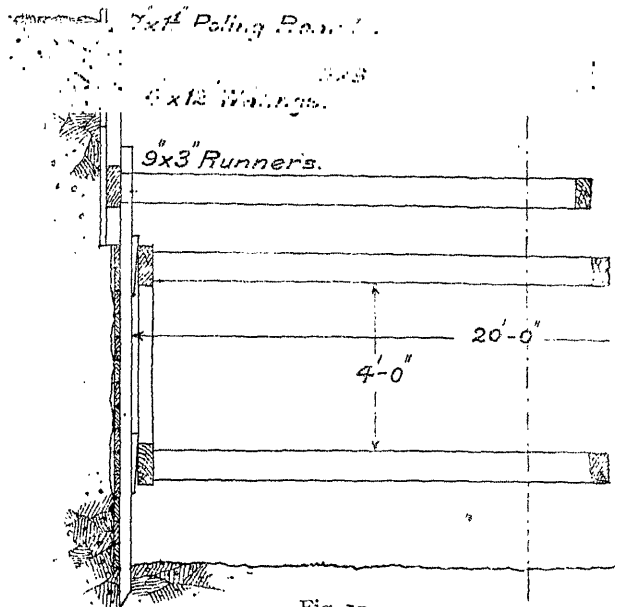


Fig. 113.

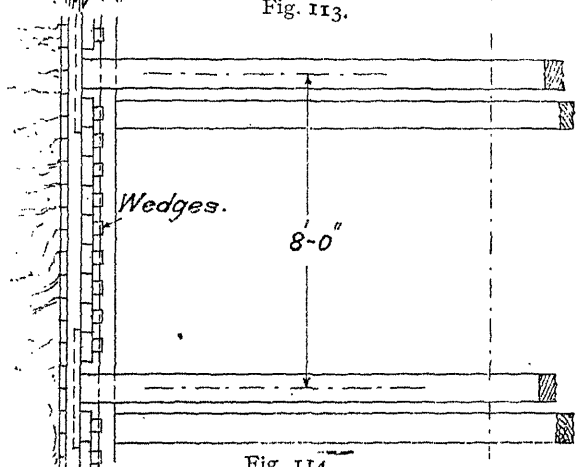


Fig. 114.

Figs. 113—114

The method illustrated in Figs. 113 to 115 is employed where the ground is soft and waterlogged, and is especially suitable for running sand. By this method as much of the earth is taken out as is possible without the sides of the excavation falling in, generally from 4 to 6 feet; this is then supported by upright sheeting, waled and strutted. The excavation is continued by lining the cutting with a

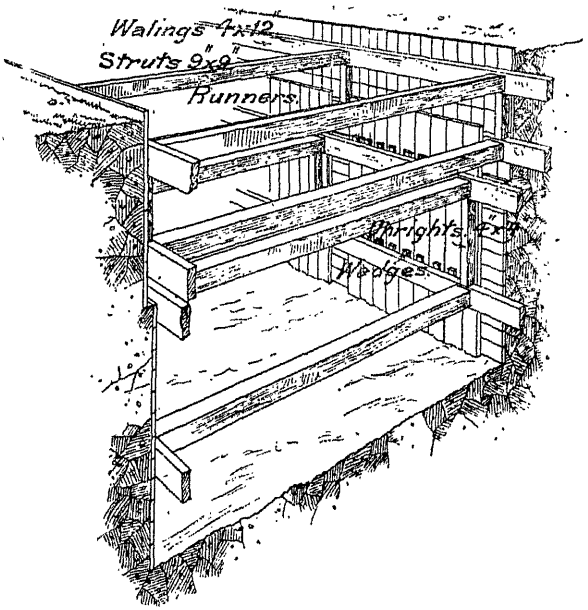


Fig. 115.

secondary system of runners, *i.e.*, battens, $7'' \times 2''$, pointed at lower ends, and of about 9 feet in length. These are waled and strutted. Between each runner and waling piece a wedge is inserted. The method of proceeding with the excavation is as follows: The wedges securing one runner are loosened, the earth from the foot removed to a depth of about 12 inches, the runner being dropped as the ground is removed and re-wedged. Each runner is successively treated in this manner till the whole system

has been lowered the necessary amount. It is essential that the feet of these runners should be at all times kept in the ground, as if any portion of the vertical side of the excavation be exposed, the earth is liable to ooze out and leave the back of the runners unsupported, and cause the whole system to collapse.

In deep trenches in loose soils, the withdrawal of the timbers must be very carefully done to prevent the collapse

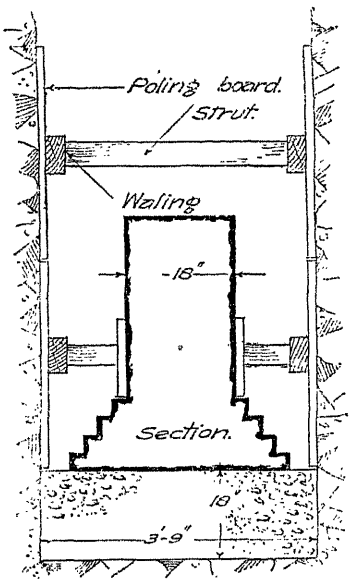


Fig. 116.

of the trench. Frequently the poling boards at the concrete level must be left in position. For the upper sections, the wall is carried up between the cross shores to a height above the poling boards in that section, short struts are then placed at intermediate positions between the main timbers on each side of the wall between the walings on one side and a short upright plate against the wall on the other, care must be taken to place these subsidiary struts exactly opposite each other on the two sides of the wall

so that the latter may not be subjected to an overturning stress. The main transverse timbers are then removed and

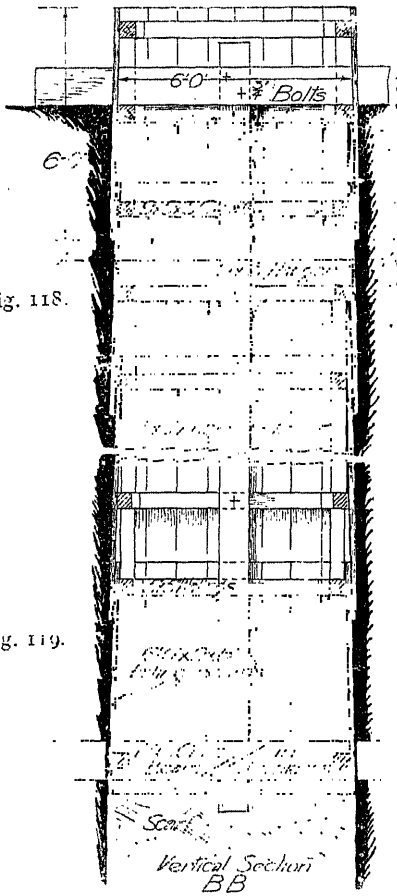


Fig. 118.

Fig. 119.

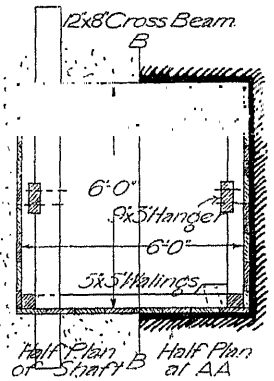


Fig. 117.

Figs. 117—119.

the spaces in the wall completed. This process is continued till the wall is above the surface level. The polings and struts are then removed in sections and the earth at once filled in to the foot of the next row of polings. The process

is continued till the whole of the timbering is removed (see Fig. 116).

Sinking Shafts.—It is often necessary to sink shafts for foundations, etc. These are made from 4 feet square and upwards, the former being the smallest size a man can work in without difficulty.

Shafts from 4 to 9 feet square are timbered as shown in Figs. 117 to 119.

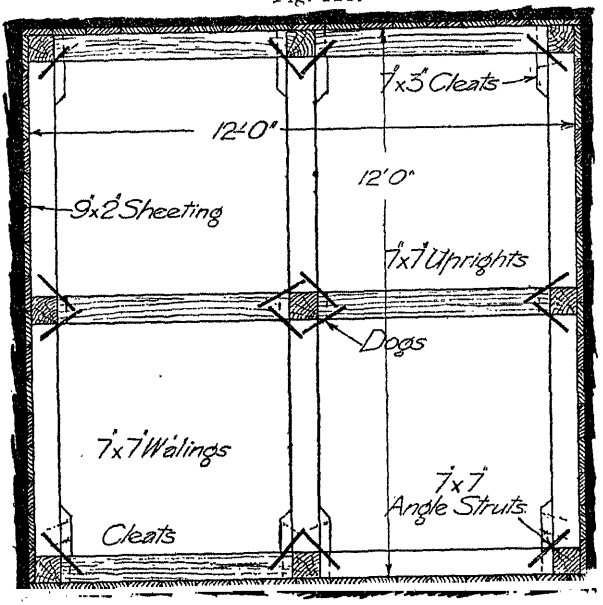
In ordinary soils the earth is excavated to a depth of at least 3 feet, and in firm soils 6 feet. The sides of the excavation are then lined with vertical sheeting, consisting of boards 9 inches wide, 1 to 1½ inches thick, strutted apart by frames of horizontal waling timbers, a pair of which are placed in position against two opposite sides, and strutted apart by another pair driven tightly between and against the remaining sides, these being secured by cleats nailed to the fixed waling pieces. Another depth of earth is then taken out, and a second system of sheeting placed in, the upper ends of which lap about 1 foot over the lower ends of the first system of sheeting; another frame is placed in position as before, securing both systems of sheeting. Uprights are fixed in the angles between the waling pieces, and often at intermediate positions along their length. This process is repeated till the required depth is obtained.

The timbering requires to be supported if the depth be great, to prevent it from sliding down on the removal of the earth from its lower end. Where this has to be done, the upper end of the shaft is left projecting about 3 feet above the ground level. The two first fixed waling timbers at the ground level are continued through the shaft, and project several feet on either side of it, a good bearing on the solid ground on both sides of the shaft being thus obtained, as shown in Figs. 117 and 118.

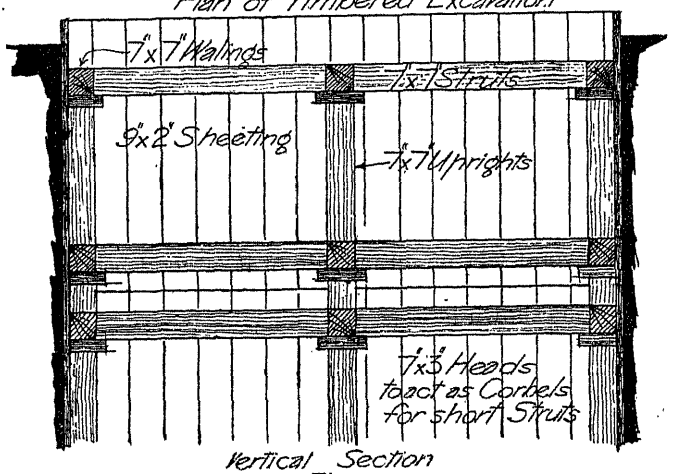
These members are usually out of balk timbers; they are strutted apart as described. An upright vertical timber is notched over this, and spiked to the face of the waling timbers below, the whole being thus tied together.

These are often supplemented by similar timbers at the bottom of the shaft. These timbers are fixed in two pieces.

Fig. 120.



Plan of Timbered Excavation



Vertical Section

Fig. 121.

Figs. 120—121.

with a scarf in the centre ; they project about 3 feet into both sides of the pit, as shown in Fig. 117. A chain is sometimes employed in addition to the timber spiked to the walings.

Intermediate struts are required to support the horizontal walings where the size of the pit is above 9 feet square. One system of struts is fixed between two opposite sides, being supported at their ends by cleats, as shown in Figs. 120 and 121, these being necessary to prevent the timbers falling should they become loose during the progress of the works. The struts that support the remaining sides intersect by butting against the first system, as shown in Fig. 121, and are therefore fixed in two pieces. The struts at their intersection are supported by uprights, on the upper ends of which short ends of timber are placed, projecting beyond the sides, acting as corbels, and forming a ledge upon which the shorter struts take a bearing, as shown in Fig. 121.

The earth is raised from the bottom of the shaft, if of a great depth, by means of hoisting tackle ; but if the cutting be shallow, stages are often erected in 6 feet heights, the earth being shovelled from one to the other till the top is reached.

Tunnelling.—In building operations it is often necessary to bore a tunnel in order to construct drains, etc., the process being carried out as follows :—

Tunnels are made just large enough for a man to work in, that is, from 4 to 7 feet square. The earth is taken out in sections of about 3 feet at a time, poling boards of the same length being then placed against the upper surface, and kept in their position by a system of strutting, consisting of a head, sill, and two uprights, out of either round or square timbers. The sill is placed in position first, being partly bedded in ground to prevent lateral motion, and being bedded in its correct vertical position by boning through from the sills previously bedded ; the head next, then the struts, which are cut and driven tightly between the two. The next section is then cleared out, commencing at the top, just enough being taken out there to allow of the next system of poling boards being inserted, these

being arranged to overlap the first system at their back end, the two being then strutted up together ; this process is repeated till the tunnel is finished.

If the soil be bad and the sides liable to fall in, they

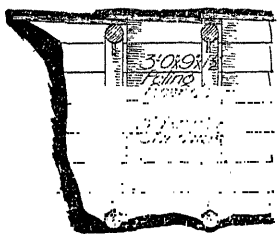


Fig. 122.

must also be lined by poling boards, these being kept in their place by the uprights.

Large spikes, similar in shape to floor brads, are driven into the head and sill, with their heads left projecting so as to be easily withdrawn, to secure the struts when in position. Wood cleats are often used in place of these.

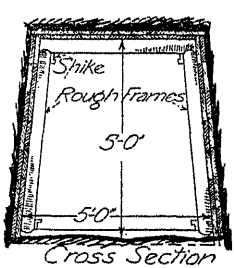


Fig. 123.

These tunnels are usually made slightly tapering from the base to the head, as shown in Figs. 122 and 123.

The excavation of basement storeys presents certain difficulties, especially if the ground is waterlogged. In every case, however, provision must be made to keep the basements dry. There are three common methods employed :—

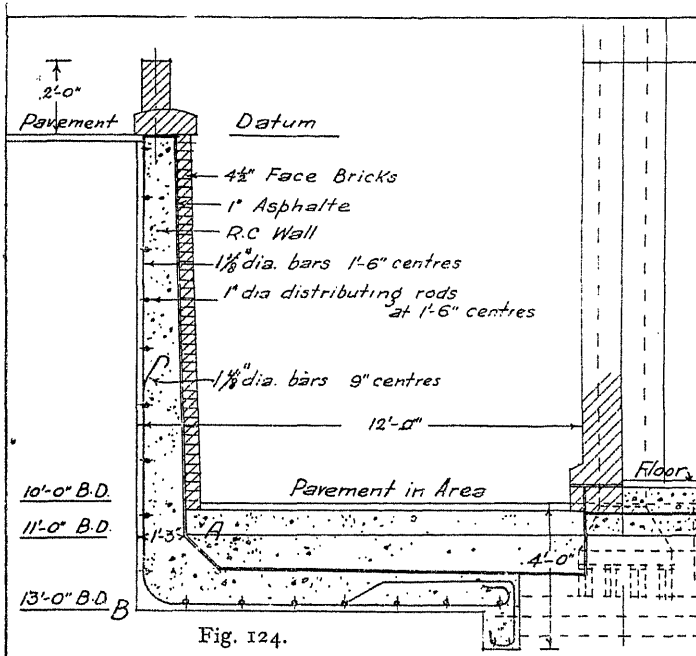


Fig. 124.

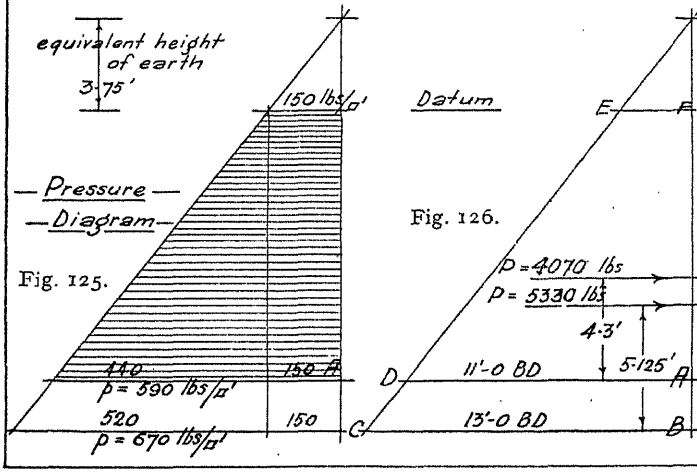


Fig. 125.

Fig. 126.

Figs. 124—126.

1. With a reinforced concrete retaining wall.
2. With steel joists encased in brick or concrete.
3. Where the wall is in close proximity to a water-course and mass concrete is employed for the wall.

The earth pressures may be obtained from Rankine's formula (see pp. 333–8). This will be a good guide, but allowances may have to be made according to the actual conditions. The same general conditions may be assumed for the three cases, with reference to the overturning pressures.

CASE 1.—Consider the case of a retaining wall with an open area, in front of the basement storey. Let the pavement be the datum level. The basement floor 10 feet below datum, and the bottom of the wall 13 feet below datum (see diagram, Fig. 124). Let the soil be clay, 120 lbs. per cubic foot. Let the angle of repose be 30 degrees. The coefficient of concrete on clay equal 0·5. In addition to the earth pressure, let there be a superload of 450 lbs. per super foot. Then the pressures "p" from Rankine's formula

$$p = w_1 h \left(\frac{1 - \sin\phi}{1 + \sin\phi}\right). \quad \text{See Table, p. 328.}$$
$$= 120h \times \cdot 33 = 40h.$$

The superload is reduced to its equivalent height of earth above datum

$$i.e.\ h_e = \frac{\text{superload}}{\text{weight of earth}} = \frac{450}{120} = 3{\cdot}75 \text{ feet},$$

and the pressure per foot super on the back of the wall

$$p_e = 40h = 40 \times 3{\cdot}75 = 150 \text{ lbs. per square foot.}$$

From the Fig. 124 the weakest point in the wall will be at the haunch at 11 feet BD at point A. Then the pressure at the point A

$$p_A = 40h = 40 \times 11 = 440 \text{ lbs. per square foot.}$$

and the pressure at 13 feet BD

$$p = 40 \times 13 = 520 \text{ lbs. per square foot.}$$

The horizontal thrust on the back of the wall will act through the centres of gravity of the trapeziums of pressure DEFA and CEFB (see Fig. 126). That through DEFA is

required to determine the moment at A, in order to obtain the thickness of the wall; that through CEFB to determine the overturning moment.

The leverage through the C.G. of trapezium DEFA = 4·3 feet
" " " " CEFB = 5·25 feet

The average pressure from DEFA

$$p = \frac{(440 + 150) + 150}{2} = 370 \text{ lbs. per square foot.}$$

The total pressure P = 370 × 11 = 4,070 lbs.
And M_p = 4,070 × 51 = 207,570 lbs. inches.

To determine the thickness of the wall using the economic ratio for steel and concrete, refer p. 728.

$$B = 137{\cdot}6\, bd^2$$

Consider a strip 12 inches wide

$$d = \sqrt{\frac{B}{137{\cdot}6 \times 12}} = \sqrt{\frac{207{,}570}{137{\cdot}6 \times 12}}$$

= 11·2 inches, say, 12 inches.

Use 1⅛ inch dia. rods, area = 0·9940 square inches.

Then $$b = \frac{A_t}{rd} = \frac{0{\cdot}9940}{0{\cdot}00893 \times 12}$$

= 9·25 inches, say 9 inches.

Then with a 2½-inch cover the thickness would be 15 inches.

As the stress diminishes towards the top, the whole of the rods need not run right up. Stop half of them at 5 ft. 0 in. from the top. Above this point the rods are at a ft. 6 in. centres. Distributing bars should be placed at about 1 ft. 6 in. centres across the reinforcing rods.

Overturning.—Let the sole plate project 9 ft. 0 in. beyond the face of the wall, that is, 10 ft. 3 in. over all (Fig. 127). Then the moment of overturning about B is

$$M_T + M_W + M_S$$

where

M_T = pressure of the trapezium CEFB acting through its centre of gravity at 5·125 feet above B.

M_W = the weight of wall multiplied by its leverage or distance from B.

M_S = the pressure of the sole plate and the filling above multiplied by the distance of its centre of gravity from B.

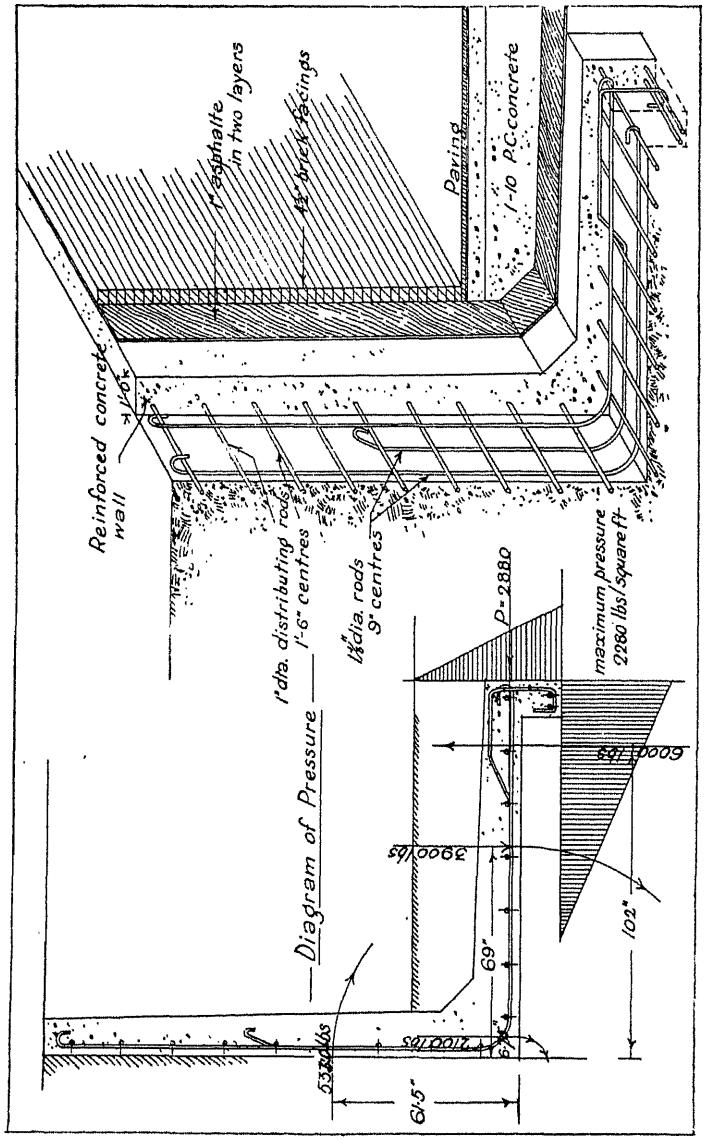


Fig. 127. Figs. 127—128. Fig. 128.

$P_T = \left(\frac{670 + 150}{2}\right)13 = 5{,}330$ lbs.

$M_T = 5{,}330 \times 61{\cdot}5'' = 330{,}000$ lbs. inches.

$W_W = \left(\frac{1 + 1{\cdot}25}{2}\right)13 \times 144 = 2{,}100$ lbs.

$M_W = 2{,}100 \times 6{\cdot}75'' = 14{,}200$ lbs. inches.

$W_S = 9 \times 3 \times 144 = 3{,}900$ lbs.

$M_S = 3{,}900 \times 69'' = 270{,}000$ lbs. inches.

$M_T + M_W + M_S = 330{,}000 + 14{,}200 + 270{,}000$
$= 614{,}200$ lbs. inches.

Equate this overturning moment to the resistance moment of the upward earth pressure multiplied by its leverage (x).

Upward earth pressure $= W_W + W_S$
$= 2{,}100 + 3{,}900$
$= 6{,}000$ lbs.

Resistance moment $= 6{,}000\,x$.

$\therefore\ 6{,}000\,x = 614{,}200$
$x = 102$ inches.

That is the centre of the upward pressure is 102 inches from B and equals $123 - 102 = 21''$ from the toe.
The distribution of the pressure is triangular, *i.e.*,

$3 \times 21 = 63'' = 5{\cdot}25'$.

Average normal pressure $= \frac{6000}{5{\cdot}25'} = 1140$ lbs. per square foot.

The maximum pressure is twice the average, *i.e.*,

Max. pressure $= 2 \times 1140$
$= 2280$ lbs. per square foot.

This is quite safe (see Fig. 127). Refer to p. 30.

Sliding.—The horizontal force $= 5{,}330$ lbs.
,, vertical ,, $= 6{,}000$,,

The coefficient of friction for masonry on clay $= {\cdot}5$. The factor of safety should not be less than 1·25.

Then $\frac{5330}{6000} = {\cdot}89$. This would be unsafe.

Taking ·4 as the safe coefficient
$\cdot 4 \times 6000 = 2{,}400$ lbs.

Leaving $5{,}330 - 2{,}400 = 2{,}930$ lbs. to be provided for (see Fig. 127).

Place a toe beam 1 foot deep, giving an area of 4 feet pressing against the earth.

Then the total horizontal pressure P

$$P = \frac{wh^2}{2}\left(\frac{1 + \sin\phi}{1 - \sin\phi}\right) \quad . \quad . \quad . \quad . \quad .$$ (See page 327).

$$= \frac{120 \times 4^2}{2} \times 3$$

$$= 2880 \text{ lbs.}$$ (See Fig. 128.)

This would under the conditions be sufficient.

CASE II.—Consider the case of a retaining wall for vaults under the pavement in a steel frame building. In this case let the earth pressure be resisted by upright rolled steel joists built in brickwork. The upper ends are connected by a channel, and the thrust at the upper ends is resisted by cross joists connected to the stancheons of the steel frame of the building (see Figs. 133 and 134). Provision must be made in the steel frame for this extra thrust (see Fig. 129). If the site is new and the ground is firm, an excavation 5 or 6 feet wide is first made to the full depth, and timbered with poling boards, walings and struts in the usual manner. The base concrete is deposited with channel to receive the lower ends of the retaining joists. A skin wall not more than 9 inches thick is then built against the earth to receive the asphalt coat, the poling boards are removed as the wall is raised. This wall must be temporarily strutted till the retaining beams are in position. The asphalt should be trowelled on in at least two coats. The retaining beams are then erected and bolted at top and bottom; the lower ends are concreted in, and they are temporarily strutted back to the skin wall. These are then built in solid with brickwork. The temporary struts for the retaining joists should be rakers. The mass of the earth inside the enclosing retaining wall can now be excavated, together with the holes for the pillar bases, and the whole side concreted and asphalted. When the first sections of the pillars have been erected with the ground-floor beams, the upper ends of the retaining beams are connected to the outer pillars with the cross beams and filler joists. The vault roof is then concreted and a shallow wall built to form a bearing for the pavement light. When this roof is asphalted, earth is filled in to support the paving slabs (see Figs. 129 to 134).

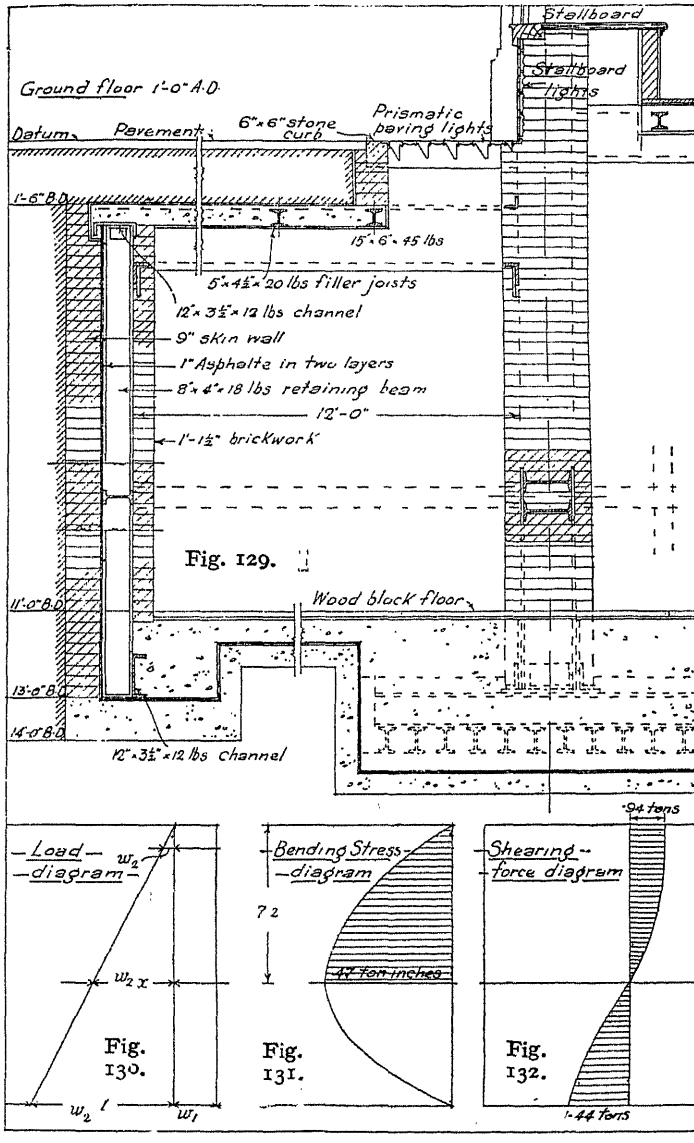


Fig. 129.

Fig. 130.

Fig. 131.

Fig. 132.

Figs. 129—132.

The retaining wall in each bay is similar to a floor slab. Take the same loading as in previous example and consider a strip of wall 1 foot wide subjected to a uniformly distributed load of w_1 or 150 lbs. per foot run, and also a load increasing from nothing at the top, by w_2 or 40 lbs. per foot run to a maximum at the bottom. Fig. 130 shows the loading diagram and Fig. 131 the bending stress diagram. The point of maximum bending moment is 7·2 feet from the top.

$$w_1 = 150 \text{ lbs./ft.}$$
$$w_2 = 40 \text{ lbs./ft.}$$
$$W_1 = 13 \times 150 = 1950 \text{ lbs.}$$
$$W_2 = \frac{w_2 l^2}{2} = \frac{40 \times 13^2}{2} = 3380 \text{ lbs.}$$
$$R_1 = \frac{W_1}{2} + \frac{W_2}{3} = \frac{1950}{2} + \frac{3380}{3} = 2102 \text{ lbs.} = 0{\cdot}94 \text{ tons.}$$
$$R_2 = \frac{W_1}{2} + \frac{2W_2}{3} = \frac{1950}{2} + \frac{2 + 3380}{3} = 3228 \text{ lbs.}$$

The point of maximum bending moment is at the point of minimum shear.

$$F = R_1 - w_1 x - \frac{w_2 x^2}{2} = 0$$
$$= 2102 - 150\,x - \frac{40\,x^2}{2} = 0.$$
$$20\,x^2 + 150\,x - 2102 = 0.$$
$$x^2 + 7{\cdot}5\,x - 105{\cdot}5 = 0.$$
$$x = \frac{-7{\cdot}5 \pm \sqrt{7{\cdot}5^2\ 4 \times 105{\cdot}5}}{2}$$
$$x = 7{\cdot}2 \text{ feet.} \quad \text{(See Fig. 132.)}$$

General expression for bending moment is

$$Bx = R_1 x - \frac{w_1 x^2}{2} - \frac{w_2 x^3}{6}.$$

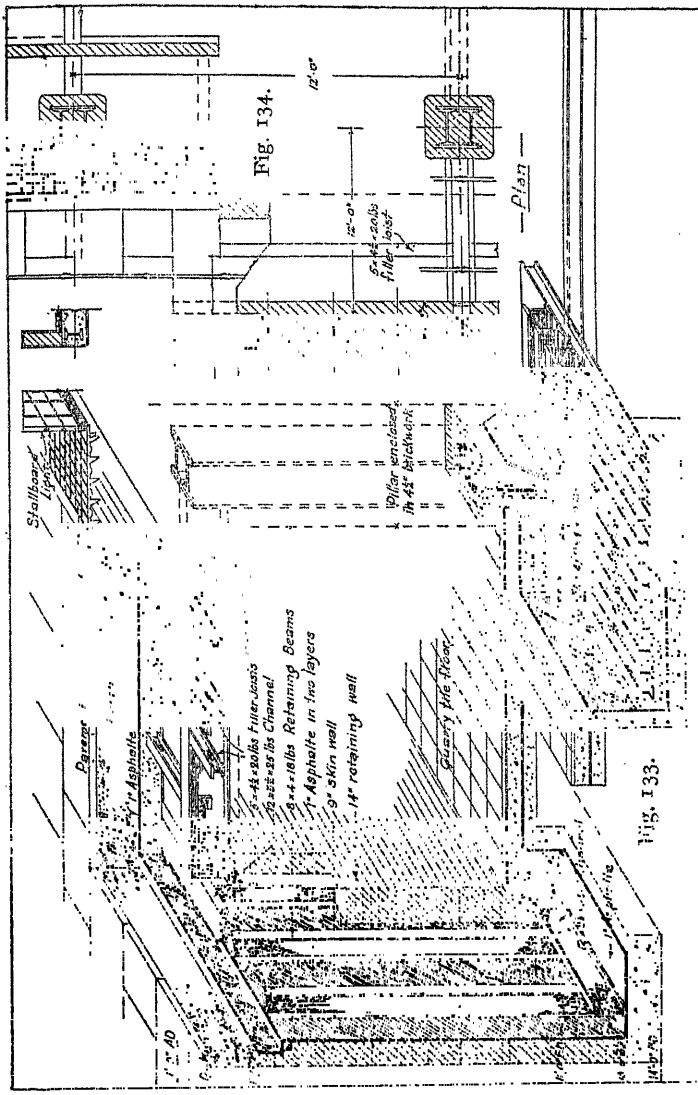
Let $x = 7{\cdot}2$ feet

$$B_{7{\cdot}2} = (2102 \times 7{\cdot}2) - \frac{150 \times 7{\cdot}2^2}{2} - \frac{40 \times 7{\cdot}2^3}{6}$$
$$B_{7{\cdot}2} = 15150 - 3890 - 2490$$
$$= 8770 \text{ lb. ft.}$$
$$= 47 \text{ ton-inches.}$$

If rolled steel joists are placed at 2 feet centres

$$B = 2 \times 47 = 94 \text{ ton-inches.}$$
$$Z = \frac{B}{y} = \frac{94}{8} = 11{\cdot}8 \text{ inches.}^3$$

use 8″ × 4″ × 18 lbs. R.S.J. at 2′ 0″ centres.


Fig. 134.
Fig. 133.
Plan
Parapet
1" Asphalte
Stallboard
Lipped
Gallery Pit Floor
9" skin wall
1" Asphalte in two layers
Pillar enclosed in 4½" brickwork
12'-0"

Figs. 133—134.

Connecting Channel.—Connecting channel at top of retaining beams. Load equals 0·94 tons per foot run. The main pillars are at 12 feet centres.

Then $B = \frac{Wl}{8} = \frac{11{\cdot}28 \times 144}{8} = 203{\cdot}125$ ton-inches.

$$B = fz \therefore Z = \frac{203{\cdot}125}{8} = 25{\cdot}4 \text{ inches}^3.$$

Use channel 12″ × 3½″ × 25·25 lbs.

Connect the bases with a similar channel (see Fig. 133).

Beam between Retaining Wall and Main Pillar.—Take the pavement load 448 lbs. per square foot. Then the load will be W = 12 × 12 × ·2 = 28·8 tons.

$$B = \frac{Wl}{8} = \frac{28{\cdot}8 \times 144}{8} = 520$$

$$Z = \frac{B}{f} = \frac{520}{8} = 65 \text{ inches}^3.$$

Use a 15″ × 6″ × 45 lbs.

Filler Beams.—Placing these at 2 feet centres.

W = 12′ × 2′ × ·2 tons = 4·8 tons.

$$B = \frac{Wl}{8} = fz \qquad \text{let} \qquad f = 9 \text{ tons}$$

$$Z = \frac{4{\cdot}8 \times 144}{8 \times 9} = 9{\cdot}6$$

Use 5″ × 4½″ × 20 lbs. Z = 10. (See Fig. 134.)

CASE III.—This method is suitable for a waterlogged site, where there is trouble with water and running soil. Under these conditions the ordinary method with poling boards would not be practicable. The method with runners (see p. 243) could be employed, but the method of steel runners, driven as sheet piling (see Fig. 137) would be preferable. Two rows of sheeting are driven at a suitable distance apart and to a depth of 1 to 2 feet below the lowest level of the foundations. The sheet piling consists of a 15 × 6 × 40 lbs.RSJ and a steel clutch of the section shown in Fig. 136, which grips the flanges of the adjoining joist sections. It is usual to fix a clutch on each pile at the works, and the two are driven together.

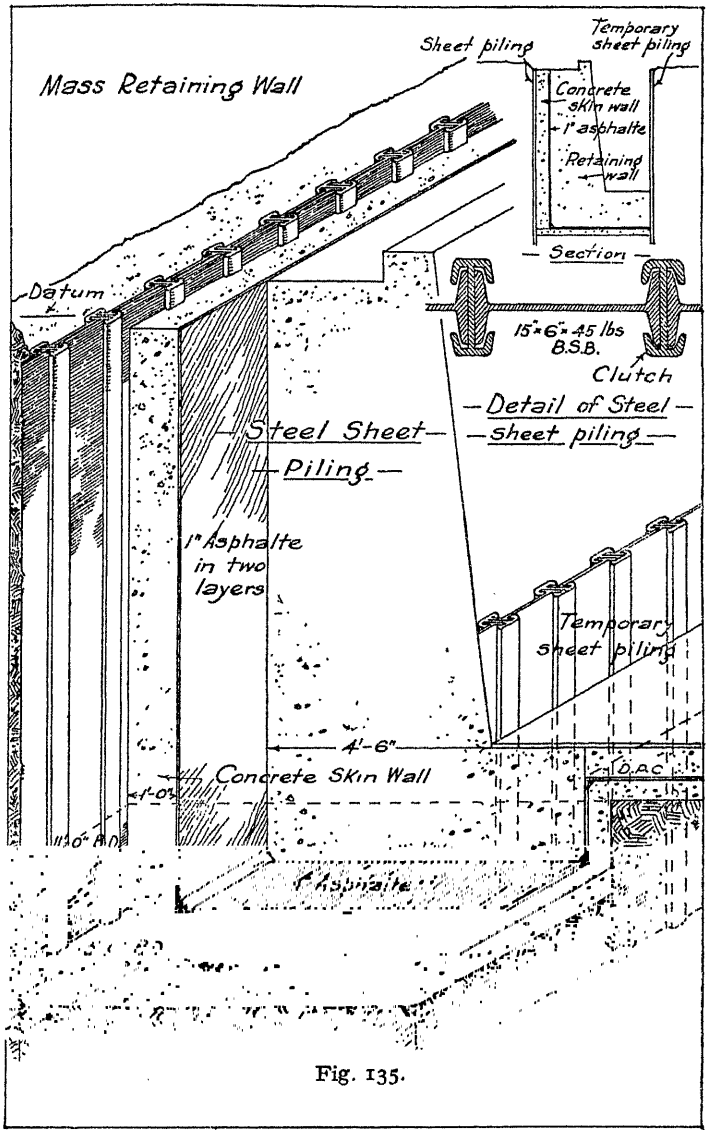


Fig. 135.

Fig. 137.

Fig. 136.

Figs. 135—137.

The earth between the piling is excavated, the sheeting being waled and strutted as the earth is removed. At the required depth the base concrete is deposited, and a skin wall of brick or concrete is built against the outer sheeting to receive the vertical asphalt. The mass concrete retaining wall is then deposited between the vertical asphalt and the formwork for the outer face of the concrete. Where there is liable to be much trouble with water, the formation of the trench is carried out in sections, to facilitate pumping operations. Transverse rows of sheeting are driven at the required intervals to form cross walls. At the ends of the first sections of the concrete walls wide vertical grooves are formed. When the final sections are deposited these grooves form vertical joggles at the junction of the sections. The transverse sheeting is withdrawn preparatory to the construction of the final sections.

When the retaining wall is completed the inner row of sheeting is withdrawn, and the concrete raft over the site is deposited and asphalted. Over the asphalt another layer of concrete to form the floor is laid, and the whole when completed forms a concrete box or enclosure in which the building is erected.

Let the point of overturning be taken at a depth of 11 feet below datum. Then the

Total side pressure = 4070 lbs.
Leverage = 4·4 feet.
Batter = 1 in 10.

$$M_P = 4070 \times 4{\cdot}4 = 17900 \text{ lb. ft.}$$

$$M_w = wh\left\{\frac{t + t - \frac{h}{10}}{2}\right\}\frac{5t}{12}.$$

$$= \frac{5\,wht}{24}\left\{2t - \frac{h}{10}\right\}$$

$$= \frac{5 \times 144 \times 11 \times t}{24}\left(2t - \frac{11}{10}\right)$$

$$= 330t\,(2t - 1{\cdot}1)$$
$$= 660\,t^2 - 363\,t.$$

$$M_w = M_P$$

$$660\,t^2 - 363\,t = 17900$$
$$660\,t^2 - 363\,t - 17900 = 0.$$
$$t^2 - 0{\cdot}55t - 27{\cdot}1 = 0$$

$$t = \frac{0{\cdot}55 \pm \sqrt{0{\cdot}303^2 + 4 \times 27{\cdot}1}}{2}$$

$$= \frac{0{\cdot}55 \pm 10{\cdot}4}{2}$$

$$= 5{\cdot}5 \text{ feet.}$$

The mass of the skin wall can be taken as part of the thickness of the wall. Then the arrangement would be as shown in Figs. 135 to 137.

Widths of Concrete Foundations.—For heavy buildings the widths of the concrete in the foundation should be determined by the bearing strength of the ground on which they rest.

The resistance of concrete to tensional stresses is negligible, the projection beyond the wall must, therefore, be so limited or the depth must be sufficiently great to prevent cantilever action. The resistance of good concrete to crushing is so high, that failure under this stress is unlikely. The shearing resistance of concrete is relatively very low, therefore the depth of concrete beds must be such as to give the maximum shearing area for any given width of the bed. It can be shown that the plane of greatest shear occurs at an angle of 45 degrees. Crushing tests on cubes of concrete generally fail by shearing along planes making angles varying between 45 and 60 degrees to the horizontal, the cube splitting into six irregular pyramids.

Let the figure ABCD (Fig. 142) be a prism subjected to a load P in the direction of its length, on any plane EF the pressure may be resolved into normal and tangential stresses.

Then $P_t = P \sin \theta$

The area of the surface EF

$$= \text{A} \sec \theta$$

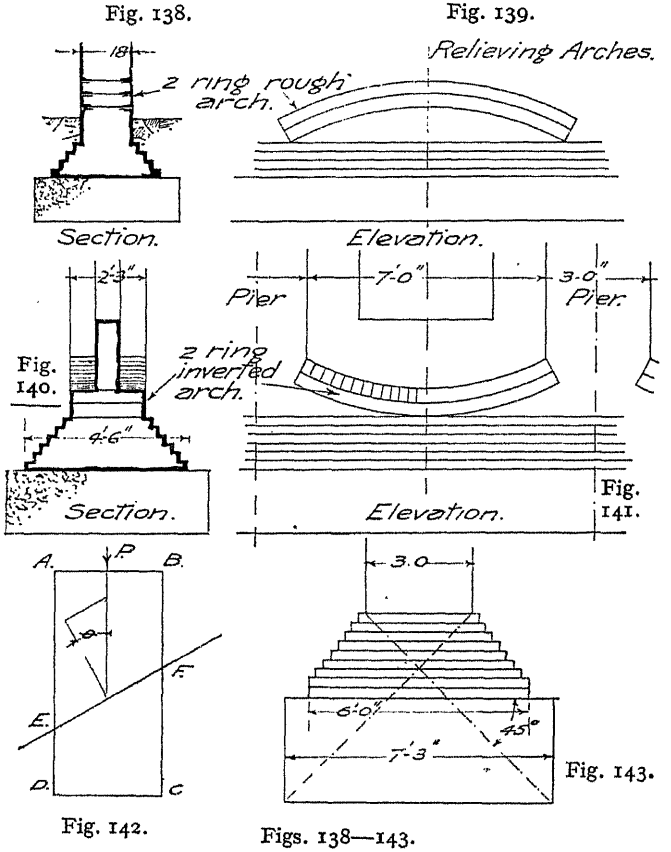
$$\therefore \quad p_t = \frac{\text{P} \sin \theta}{\text{A} \sec \theta} = p \sin \theta \cos \theta$$

$$= \frac{p}{2} \sin 2\theta$$

in like manner $p_n = \dfrac{\text{P} \cos \theta}{\text{A} \sec \theta} = p \cos^2 \theta$

It will be found that p_t is a maximum when $\theta = 45°$.

As the foundation concrete receives considerable support from the adjacent earth, it will usually be sufficient to determine the depth of the concrete by drawing lines at



Figs. 138—143.

45 degrees from the base of the wall to intersect the side of the concrete bed as shown in Fig. 143.

The following illustrates the application of this theory to piers :—

EXAMPLE: Calculate the necessary dimensions of brick footings, and the width and depth of concrete for a square brick pier of 3 feet side, stressed to 5 tons per square foot of section, the safe loads of lias lime concrete and earth being taken as 3 and 1 tons per superficial foot respectively.

The area of the base of the footings in feet will be the total load divided by the safe load per super foot of concrete. Let the weight of the brick footings be taken as approximately 1 ton, then—

$$\frac{\text{Total load}}{\text{Safe load upon concrete}} = \frac{45 + 1}{3} = 15\tfrac{1}{3} \text{ feet super.}$$

The side of base footings will therefore equal in nearest brick dimensions 4 ft. $1\frac{1}{2}$ in.; but it is usual to make the side of lowest course of footings twice the width of the pier, that is in this case 6 feet.

The area of the base of the concrete in feet will be the total load divided by the safe load sustained per super foot by the earth. Let the weight of the concrete be taken as approximately 5 tons, then—

$$\frac{\text{Total load}}{\text{Safe load upon earth}} = \frac{45 + 1 + 5}{1} = 51 \text{ feet super,}$$

and the side of base of concrete will therefore equal 7·14 feet, say 7 ft. 3 in., as shown in Fig. 143.

Fig. 143 shows the footings and concrete to satisfy the preceding calculations and the bye-laws, the brick footings being splayed at the usual angle, and the depth of concrete being determined by drawing lines at 45 degrees from base of wall and the point of intersection with the calculated width of concrete, will give the depth, as previously explained. Fig. 143 shows how the brick footings are usually arranged in practice, which, in this case, may be supposed to be substituted for the equivalent depth of concrete.

Determination of the Resistance of Soils.—The absolute value of the resistance of any soil is indeterminate, but there are two methods of obtaining an approximate value. First, by driving a square-ended pile a short distance into the surface to be tested and noting the distance it sinks under the impact of a heavy weight. Secondly, by dropping

a heavy iron bar with a head shaped like a rammer with a flat end of a known area, through a given height and measuring the depth of the impression. Both methods are based on the same principle, and the measure of the resistance can be estimated from the formula given on p. 280,

$$i.e., \quad R = \frac{Wh}{d}$$

EXAMPLE : A weight of 40 lbs. with an area of 36 square inches is dropped from a height of 10 feet and makes an impression $\frac{1}{2}$ inch in depth.

Then $$R = \frac{40 \times 120}{\frac{1}{2}} = 9600 \text{ lbs.}$$

And R per square ft. $$= \frac{9600 \times 4}{2240}$$

$$= 16 \text{ tons per square foot.}$$

Taking a factor of safety of one-eighth would give a safe resistance of 2 tons per square foot. The weight should be dropped on the bottom of the foundation trench, not on the surface.

The resistance of any soil increases as its depth from the surface is increased. It is obvious that if a pier or wall settles, the earth immediately under it must be displaced. The resistance to displacement may be obtained from Rankine's theory of principal stresses, which is fully explained on pp. 328–9.

Depth of Foundations.—For the purpose of calculating the depth of foundations in a yielding soil required for, say, a wall or pier, use is made of formula (7), pp. 323–329.

$$\frac{f_x}{f_y} = \frac{1 + \sin \phi}{1 - \sin \phi}$$

f_x and f_y are the principal stresses in the material, and ϕ is the angle of repose, and f_x is greater than f_y.

With reference to Fig. 144, consider a particle of earth at position (1). It is subject to two principal stresses f_1 and f_2. At a point (2), there is the same horizontal stress f_2 and another vertical stress f_3.

Now $$f_1 > f_2$$

$$\therefore \quad \frac{f_1}{f_2} = \frac{1 + \sin \phi}{1 - \sin \phi} \quad \ldots\ldots\ldots \quad (1)$$

At (2) the greater stress will be f_2. Hence

$$\frac{f_2}{f_3} = \frac{1 + \sin \phi}{1 - \sin \phi} \quad . \quad . \quad . \quad . \quad (2)$$

Let w_1 = weight per cubic foot of earth.
Let d = minimum depth of foundation.

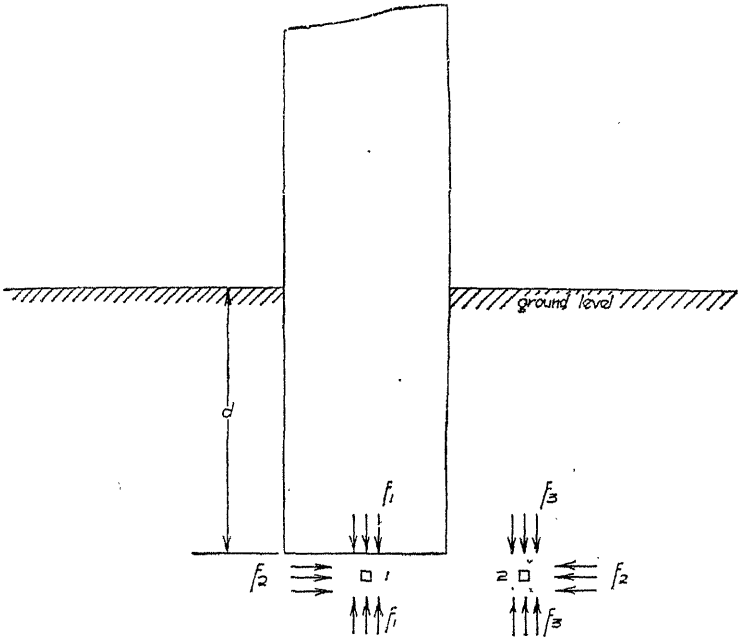


Fig. 144.

Then from (1)

$$f_2 = f_1 \frac{1 - \sin \phi}{1 + \sin \phi}$$

and from (2)

$$f_2 = f_3 \frac{1 + \sin \phi}{1 - \sin \phi}$$

$$\therefore \quad f_1 \frac{1 - \sin \phi}{1 + \sin \phi} = f_3 \frac{1 + \sin \phi}{1 - \sin \phi}$$

Now $$f_2 = w_1 d$$

$$\therefore \quad w_1 d = f_1 \left\{\frac{1 - \sin \phi}{1 + \sin \phi}\right\}^2$$

$$d = \frac{f_1}{w_1}\left\{\frac{1 - \sin \phi}{1 + \sin \phi}\right\}^2 \quad . \quad . \quad . \quad . \quad . \quad (3)$$

The above formula gives the minimum depth of foundation required. The stress f_1 is easily calculated if the loading

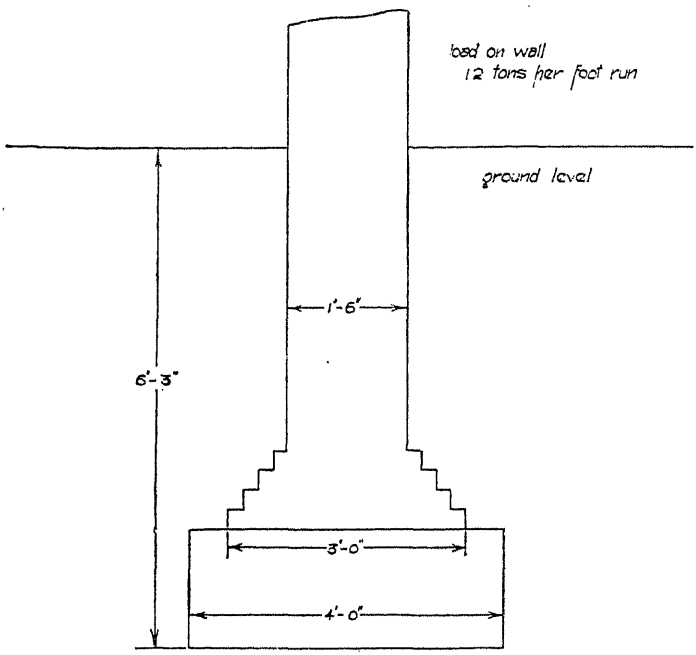


Fig. 145.

on the wall is known. Note carefully that if w_1 is in pounds per cubic foot, then f_1 is in pounds per square *foot.*

EXAMPLE 1 (Fig. 145).—Let it be required to find the depth of foundation required for a wall having the dimension shown in the figure and carrying a load of 12 tons per foot run. Let the angle of repose of the earth be 30 degrees, and the weight of the earth 120 lbs. per cubic foot.

Consider one foot length of the wall. Then the load is spread over an area of $4' \times 1'$.

Hence $$f_1 = \frac{12 \times 2240}{4} = 6720 \text{ lbs./sq. ft.}$$

$$\therefore \quad d = \frac{6720}{120}\left\{\frac{1 - 0{\cdot}5}{1 + 0{\cdot}5}\right\}^2$$

$$= 6{\cdot}22 \text{ ft.}$$

Hence take the depth as $6'\ 3''$.

Concrete Foundations and Inverted Arches.—Where the ground is yielding and the loads are concentrated by the piers at intervals on the foundations, the piers may be connected by inverted arches, which serve to distribute the pressure uniformly along the whole length of the foundation. The width of the arches should be equal to the full width of the piers, and the most suitable segment for the arch to distribute the pressure uniformly is one having a rise of one-eighth of the span. Figs. 140 and 141 illustrate an inverted arch.

Steel and Concrete Foundations.—Where heavy loads are transmitted to (1) a stratum of low bearing value, or (2) where it is desirable to avoid the use of the deep beds of concrete required to eliminate tensional stresses in the foundation, it is a common practice to extend the area over which the loads are distributed by the employment of either timber or steel. These materials having a high tensional value, render it possible to have a maximum lateral extension of the foundation with a minimum of depth. Timber should only be employed, where there are reasons to assume that it will be either constantly wet or constantly dry, alternation to wetness and dryness quickly causes the timber to rot. Steel must be embedded in a rich water-resisting cement concrete; when thus enveloped it is preserved from corrosion.

Timber or steel employed in this manner forms a raft, the walls, piers, or stanchions being concentrated in the centre, leaving the timber or steel sections projecting the requisite distance beyond the footings or bases and forming inverted cantilevers on each side of the wall or pier.

In the computation of foundations of this type it is necessary first to determine the bending moments on the cantilevers. There are two possible cases, see Figs. 146—147.

(1) Where the portion of the beam that is below the wall or stanchion is not fixed and, as a consequence, would bend, the maximum moment would occur at the centre of the beam.

(2) Where the portion of the beam below the wall or stanchion may be considered fixed, in this case the maxi-

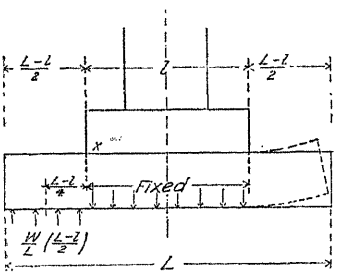


Fig. 146.

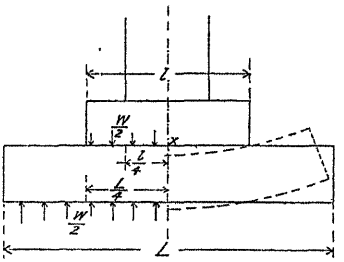


Fig. 147.

mum moment may be taken at the edge of the base, or such point as may be considered the point of fixing.

Case 1 should be taken when dealing with brick or stone walls or piers; Case 2 only where the central portion could be regarded as rigidly fixed to the base directly above. In practice, however, Case 1 is invariably taken.

Let l = the width of base
L = the length of the foundation beam.
W = the total load.

Then Case 1, Fig. 147.—Consider the load on one side of the centre line, and the BM at the point x in the centre of the beam; $\frac{W}{2}$ will be distributed over the base and pressing downwards with a leverage of $\frac{l}{4}$. The resistance

of the earth on one side of the centre will be $\frac{W}{2}$ pressing upwards with a leverage of $\frac{L}{4}$. Therefore—

$$B_x = \left(\frac{W}{2} \times \frac{L}{4}\right) - \left(\frac{W}{2} \times \frac{l}{4}\right)$$
$$= \frac{W}{8}(L - l) \qquad (1)$$

Case 2, see Fig. 146.—The central portion of the beam below the base is fixed, and the pressure of the load is transmitted through it to the ground direct. Then the BM at x at the edge of the base is deduced as follows: The length of the cantilever beyond $x = \frac{L - l}{2}$. The upward pressure of the earth over this length $= \frac{W}{L}\left(\frac{L - l}{2}\right)$ and is acting with a leverage of $\left(\frac{L - l}{4}\right)$. Therefore—

$$B_x = \frac{W}{L}\left(\frac{L - l}{2}\right)\left(\frac{L - l}{4}\right)$$
$$= \frac{W}{8L}(L - l)^2 \qquad (2)$$

Plank Foundations.—These may be used in soft and wet soils where a suitable timber is plentiful and where steel and concrete are not readily obtainable. They consist of a raft formed of two layers of planking, the lower one laid transversely and the upper layer longitudinally to the direction of the wall. The planks in each layer are spiked to each other and the upper layer to the lower. Timber for this purpose should be treated to one of the preservative processes, preferably creosoting.

EXAMPLE.—Let it be required to support a wall 2 bricks in thickness and stressed with a load of 5 tons per superficial foot by a plank foundation, the safe resistance of the soil being $\frac{3}{4}$ ton per superficial foot. Determine width of foundation and thickness of planking required.

Taking a length of 1 foot—

$$\therefore \quad \text{width of foundation} = \frac{\text{total load}}{\text{safe resistance of earth}}$$

$$,, \quad ,, \quad = \frac{5 \times 1\frac{1}{2}}{\frac{3}{4}}$$

$$,, \quad ,, \quad = 10 \text{ feet.}$$

As the length of the transverse planking beneath the

Fig. 148.

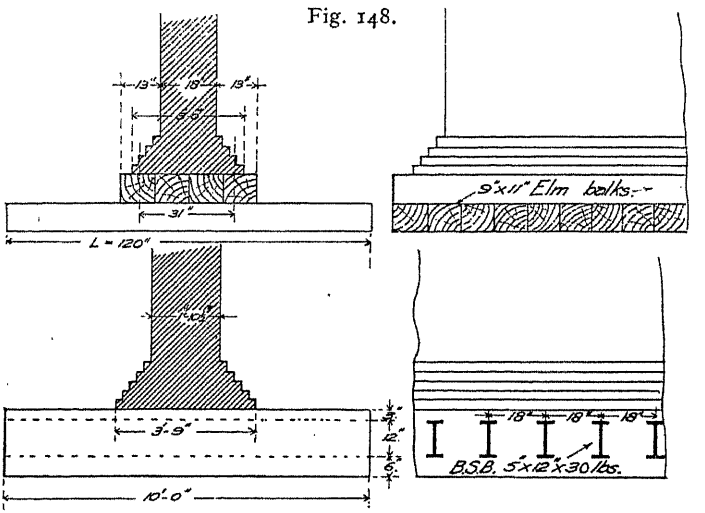


Fig. 149.

Figs. 148—149.

wall is not fixed, Method (1) for determining the bending moment would be employed. Let elm be selected for the planking. Then the value of f for elm = 2·68 tons per square inch, and with a factor of safety of $\frac{1}{5}$ = ·556 tons sq. in. W = 7·5 tons. Let l = the average width of the projection of the footings and longitudinal planking plus the thickness of the wall = 2′ 7″.

$$\text{Then} \quad \frac{W\,(L - l)}{8} = \frac{fbd^2}{6}$$

$$\frac{7{\cdot}5\,(120 - 31)}{8} = \frac{{\cdot}556 \times 12 \times d^2}{6}$$

$$\text{and } d = \sqrt{\frac{7{\cdot}5 \times 89 \times 6}{8 \times 12 \times {\cdot}556}}$$
$$= 8{\cdot}7, \text{ say } 9 \text{ inches.}$$

See Fig. 148.

EXAMPLE.—A wall 1 ft. $10\frac{1}{2}$ in. in thickness supports a load of 15 tons per lineal foot. The width of footings at base to comply with bye-laws is 3 ft. 9 in. The safe bearing strength of earth is $1\frac{1}{2}$ tons per superficial foot; therefore—

$$\text{the width of foundation} = \frac{\text{total load}}{\text{safe resistance of earth}}$$
$$= \frac{15}{1\frac{1}{2}} = 10 \text{ feet.}$$

Let steel joists embedded in concrete at 1 ft. 6 in. centres be employed to distribute the pressure over the width of the foundations. Let $W = 15 \times 1{\cdot}5 = 22{\cdot}5$ tons, $L = 120$ inches and l = the width of the footings at base — one projection, *i.e.*, $l = 3'\ 9'' - 11\frac{1}{4}'' = 45'' - 11\frac{1}{4}'' = 33\frac{3}{4}''$, say 33 inches. Then by Method (1)

$$\frac{W\,(L - l)}{8} = fZ$$
$$\frac{W\,(L - l)}{8f} = Z$$
$$\frac{22{\cdot}5\ (120 - 33)}{8 \times 8} = 30{\cdot}6.$$

Select an R.S.J., $12'' \times 5'' \times 30$ lbs. $Z = 34{\cdot}49$. This is the beam having its modulus nearest to that required. See Fig. 149.

Grillage.—In cases where the weight is transmitted by piers or stanchions the foundation is extended on all faces of the pier. This necessitates the system being built up in two tiers. The arrangement is termed a grillage. See Figs. 150—152.

The B.S.S. No. 449 provides for a considerable increase in the normal working stresses for grillage beams amounting to 50 per cent. in the case of mild steel and $33\frac{1}{3}$ per cent. in the case of high tensile steel provided that :—

Fig. 151.

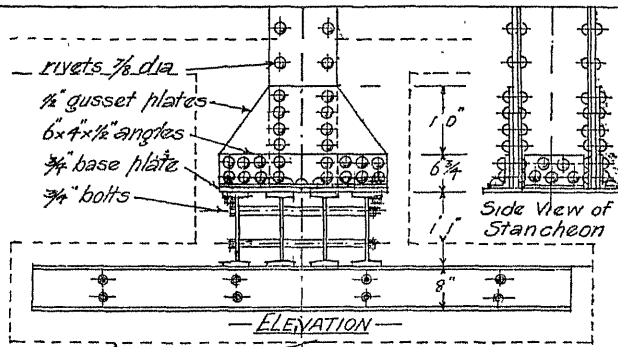


Fig. 150.

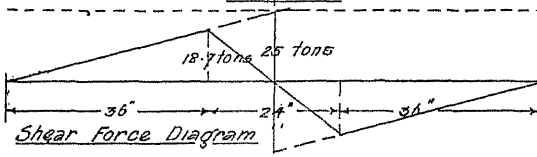


Fig. 151a.

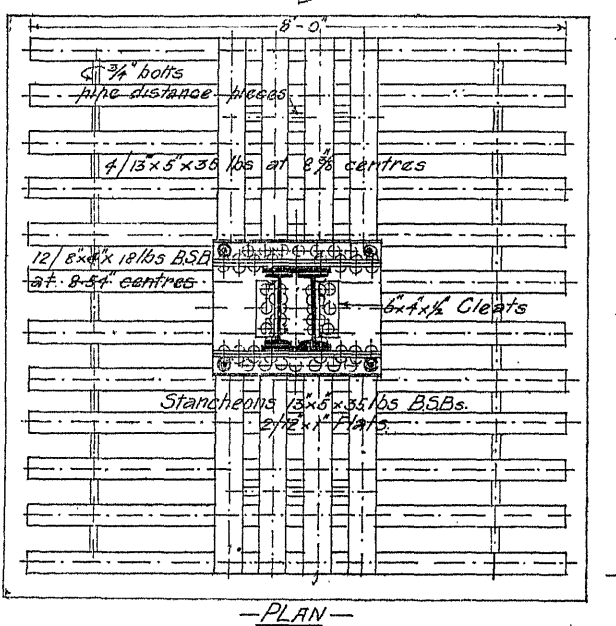


Fig. 152.

Figs. 150—152.—Two Tier Steel Grillage Foundation.

(*a*) Such beams are so enveloped in a fine concrete (*q.v.*) that their entire surface is in close contact with such concrete except where they are in direct contact with and transverse to one another.

(*b*) The beams are spaced apart not less than 3 inches and the concrete solidly tamped round them.

(*c*) The thickness of the concrete above the upper flange of any tier is not less than 4 inches.

(*d*) The thickness of concrete at the outer sides of the external beams is not less than 4 inches beyond the external edges of the flanges.

EXAMPLE.—Let a steel stanchion 15″ × 12″ overall be formed with 2 No. R.S.J. 13″ × 5″ × 35 lbs. and 2 No. flats 12″ × 1″, one on each flange. The load carried is 200 tons. Let the safe resistance of the soil be 3 tons per square foot. Safe longitudinal stress (*f*) in steel will be (8 + 50 per cent.) = 12 tons/inch.²

$$\text{Area of base required} = \frac{200}{3} = 66 \text{ square feet.}$$

say, 8′—0″ × 8′—0″.

Calculate upper tier

$$fZ = M \qquad \text{(p. 565)}$$

$$= \frac{W}{8}(L - l)$$

$$\therefore \quad Z = \frac{W(L - l)}{8f} = \frac{200(96 - 24)}{8 \times 12}$$

$$= 150 \text{ in.}^3$$

Try 4 No. beams. Z required for each one is

$$\frac{150}{4} = 37{\cdot}5 \text{ in.}^3$$

Use 4 No. 13″ × 5″ × 35 lbs. R.S.J. for top tier.

It is necessary with grillage beams to check the strength of the web for resistance to buckling. (See similar example, p. 586.) Each beam will carry 50 tons.

$$\text{Max. shear (Fig. 151a)} = \frac{50}{2} \times \frac{36}{48}$$

$$= 18{\cdot}7 \text{ tons.}$$

Assume that this is distributed uniformly over the

gross sectional area of the web which, according to B.S.S. 449, shall be calculated on the full depth of the beam.

$$\text{Max. shear stress} = \frac{18{\cdot}7}{13 \times 0{\cdot}35} = 4{\cdot}12 \text{ tons/in.}^2$$

Consider a strip of web at 45 degrees to the horizontal. Calculate the safe stress on this as a pillar. This stress should not be greater, numerically, than the shear stress calculated above.

Net depth of web $= 13 - (2 \times 0{\cdot}604) = 11{\cdot}8$ inches.

Length of strip at 45° $= 11{\cdot}8\sqrt{2} = 16{\cdot}7$ inches.

$$\text{Radius of gyration} = r = \sqrt{\frac{I}{A}} = \sqrt{\frac{\frac{bd^3}{12}}{bd}} = \sqrt{\frac{d^2}{12}}$$

d = web thickness in this case.

$$r = \sqrt{\frac{0{\cdot}35^2}{12}} = 0{\cdot}10,$$

$$\frac{l}{r} = \frac{16{\cdot}7}{0{\cdot}10} = 165$$

From Rankine's curves (Fig. 392) allowable stress $= 3{\cdot}65$ tons/in.^{2} This is a little below the value for maximum shear stress calculated above; but a 10 per cent. increase in the allowable stress would bring the two figures about the same. As the webs are encased in concrete this is permissible.

The joists are spaced at 8-inch centres, leaving 3 inches clear between each joist.

Calculate lower tier

$$Z = \frac{W(L - l)}{8f} = \frac{200\ (96 - 29)}{8 \times 12}$$

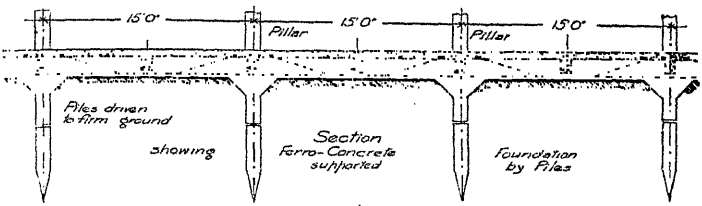
$$= 140 \text{ ins.}^3$$

Try 12 No. beams, $Z = \frac{140}{12} = 11{\cdot}7$ in.^{3}

12 No. 8″ × 4″ × 18 lbs. R.S.J. spaced out to 8′—0″ will leave approximately $4\frac{1}{8}$ inches clear between each joist.

Check the web for buckling as before.

Fig. 153.



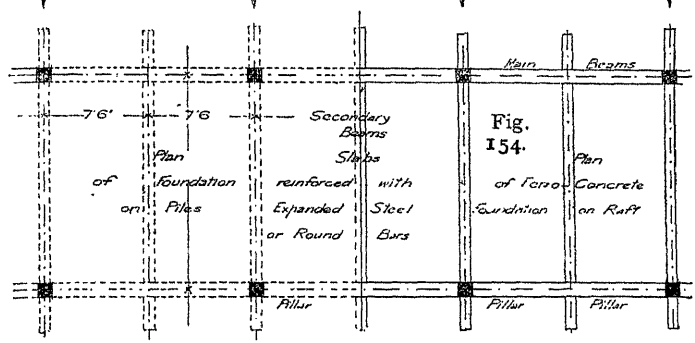
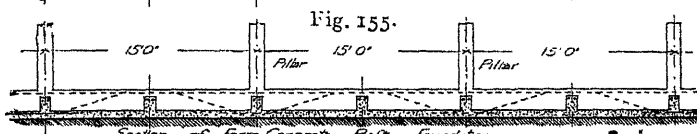
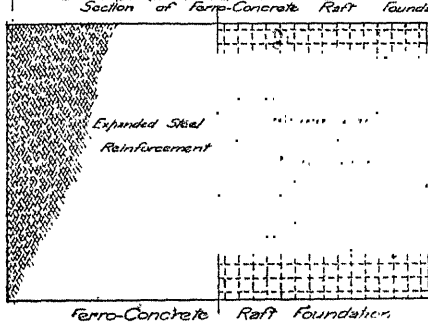


Fig. 155.





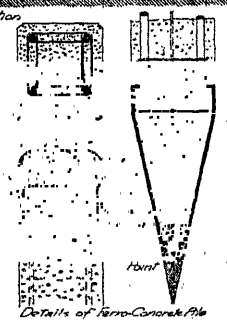


Fig. 156. Fig. 157. Fig. 158.

Figs. 153—158.

Ferro-concrete Foundations.—Reinforced foundations suitable for heavy buildings on soft soils may be constructed in one of three ways—first, on piles supporting piers; secondly and thirdly on rafts. In the first place the ferro-concrete piles are driven in immediately under the piers, the latter forming a continuation of the piles; on the pile heads a floor is constructed similar to those of the upper floors as shown in Figs. 153—154. In the second case on soils having a uniform resistance the concrete raft is constructed as an inverted floor, by first placing a reinforced concrete slab over the whole site, this being stiffened by main girders which support the piers, and also by secondary girders to stiffen the floor slabs, the whole being constructed similar to an ordinary ferro-concrete floor, but inverted, as shown in Figs. 154—155. In the third case, on soils having a non-uniform resistance, a slab of ferro-concrete of uniform thickness is placed over the whole site, the reinforcement being of rods, arranged lattice-wise, or of expanded metal and placed about the centre of the thickness, as shown in Figs. 156—157.

(1) *Pile Foundations.*—The foundations are formed by driving piles, which may be of timber or ferro-concrete from 9 inches square and upwards, till their points rest on the solid ground, or till the friction against their sides is sufficient to safely sustain the load. Timber or ferro-concrete piles are driven in at varying distances apart under all the piers of the building; the timber piles should be connected by inverted arches, as shown in Fig. 164, to distribute the pressure uniformly along the foundation, and the piles being bridged both transversely and longitudinally by horizontal timbers of about the same sectional area as the piles, the whole being arranged as shown in Fig. 164, which also illustrates the arrangement of the piles at the angle of a building. On the top of the timbering may be placed a platform of timber or a layer of concrete, on which the walls are built. Ferro-concrete piles are connected by ferro-concrete beams, as shown in Fig. 153.

Pile Driving.—Piles are driven in the ground usually by means of a ram, which is a block of iron, sometimes called a monkey, weighing from 5 to 40 cwts., raised by a

crab winch, actuated by manual or steam power. Fig. 159 is an illustration of a pile engine, capable of being worked by manual or by steam power. The monkey in these machines is raised to a given height, when, by an arrange-

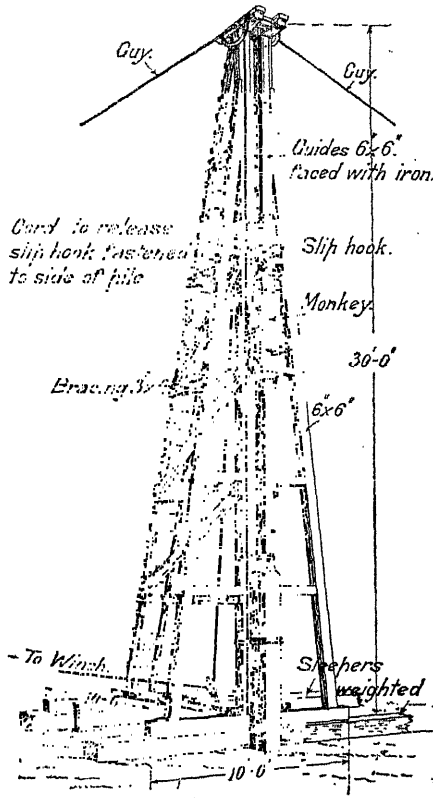


Fig. 159.

ment known as a slip-hook, it is released, and descends with a force increasing directly as the height to which it had been raised. This distance ranges from 5 to 10 feet, usually 5 feet, as a comparatively heavy monkey with a shorter fall is found practically to be better than a light

monkey with a great fall, the latter having the tendency to shiver the pile instead of forcing it downwards. Piles are considered to be sufficiently driven when the last blow does not sink the head more than $\frac{1}{4}$ inch.

The supporting power of a pile depends—(*a*) upon the resistance at the point to penetration; (*b*) the friction of the earth upon the sides of the pile; and (*c*), if projecting above the ground, its strength as a pillar.

The following empirical and well-known formula of Major Saunders, U.S. Eng., will give the safe load supported by piles in cwts. :—

Let d = the distance driven by last blow in inches
h = height fallen by monkey in inches
W = weight of monkey in cwts.

$$\text{then safe load in cwts.} = \frac{\text{W}h}{8d}.$$

EXAMPLE.—Determine the supporting power of a pile, which at the last blow of a weight of 10 cwts. falling freely through the height of 10 feet is urged through the ground a distance of a quarter of an inch.

Then by the formula

$$\text{S L} = \frac{\text{W}h}{8d}$$

$$\text{S L} = \frac{10 \times 120}{8 \times \frac{1}{4}}$$

$$\text{S L} = 600 \text{ cwts.}$$

that is, 30 tons can be safely carried by every such pile. It can be shown by the laws of falling bodies that the factor of safety taken by Major Saunders is one-eighth of the ultimate resistance, for

$$\text{Force or resistance} = \text{mass} \times \text{acceleration}$$

$$\therefore \quad \text{R} = \text{M}\,a$$

$$\text{but acceleration} = \text{velocity}^2 \div 2 \text{ space}$$

$$\therefore \quad \text{R} = \frac{\text{M}\,v^2}{2\,\text{S}}$$

but S is in this case the distance passed through by a body when arrested at its greatest velocity till it comes to rest

$$\text{but mass} = \text{weight} \div \text{gravity}$$

$$\therefore \quad \text{R} = \frac{\text{W}\,v^2}{2g\,\text{S}}$$

but $v^2 = 2g$ S, and S here equals h or height passed through by a falling body from its state of rest till the instant it strikes the pile.

$$\therefore \quad R = \frac{W\ 2gh}{2g\ S}$$

$$\text{that is} \quad R = \frac{Wh}{S}$$

but S in this case is the d of Major Saunders' formula, being the distance that the pile is urged by last blow

$$\therefore \quad R = \frac{Wh}{d}$$

Small steam hammers are sometimes used as pile-driving machines. Rankine says that it appears from practical examples that the limits of the safe loads on piles are as follows :

1. In piles driven till they reach the firm ground, 1,000 lbs. per square inch of area of head.

2. In piles standing in soft ground, by friction, 200 lbs. per square inch of area of head.

Ferro-Concrete Piles.—Of late years piles, usually of circular or square section, have been most successfully made of concrete, reinforced with steel. Fig. 158 illustrates a pile, 14″ × 14″. Ferro-concrete is now rapidly displacing the use of timber for piles. These when properly constructed are imperishable. They are of concrete reinforced with steel rods, the latter giving to these piles sufficient tenacity to resist fracture induced by the blows of the pile driver. A further description of these is given in the article on ferro-concrete.

Stewarts (Cast in Place).—Concrete or ferro-concrete piles. These consist of driving a steel tube into the ground to the depth required, and inside this to cast the pile.

The manufacturers describe these as follows :—" A heavy steel tube, generally 16 inches outside diameter, is placed on top of a loose cast iron point and driven into the ground (until the required final set is obtained) by means of a heavy drop hammer operated by a high speed friction winch. The tube is filled with concrete to a height

of several feet (according to length of pile) above the level at which it is desired to finish the head of the pile. A powerful pulling tackle is coupled on, and the tube is slowly and steadily drawn out of the ground. As the tube is withdrawn the concrete sinks and expands, filling up tightly the hole so formed. Prior to depositing the concrete reinforcement of any desired type may be placed inside the tube. Under normal conditions, no reinforcement is used."

During the last few years large numbers of these piles have been used throughout the United Kingdom of lengths varying from 10 to 70 feet. A large proportion of this work has been done under a guarantee of stability. The cast in place costs less than cast and driven piles, and frequently less than timber without taking into account the much shorter life of a timber pile. The cast in place pile has the great advantage that the concrete never receives a blow, and there is no danger of its being shattered underground nor broken away from its reinforcement.

Soft Soils.—The following are cases that require special treatment :—(1) Soft soils of a great depth ; (2) soft soils with hard strata beneath ; (3) soils not having a uniform resistance, formed of rocks which have hollows or fissures filled up with some softer material.

1. Foundations in the first case may be made in one of two ways, or by a combination of both—(*a*) by sheet piling ; (*b*) by forming the foundation on planks or by a concrete raft.

(1) *Sheet Piling.*—Sheet piling, as shown in Figs. 160 and 161, is used to prevent the lateral escape of the soft soil. It consists of flat timbers, about 9″ to 11″ × 3″, driven in and enclosing the site to be built upon, the area of the latter being sufficient to withstand the pressure brought to bear upon it ; as the soil cannot escape, it must necessarily remain and support the structure. If the site is to be drained it must be done before the building is erected.

In order to enclose a site with sheet piling, it is necessary to drive guide piles into the soil, at intervals of from 6 feet to 10 feet apart. These usually consist of timbers 9 inches square and upwards, pointed and shod with iron at the

Fig. 160.

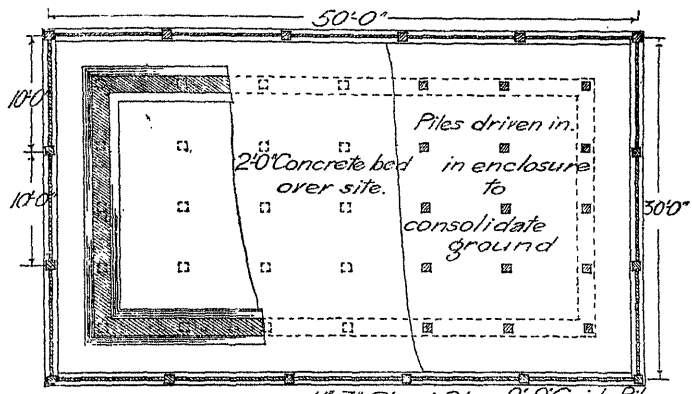


Fig. 161.

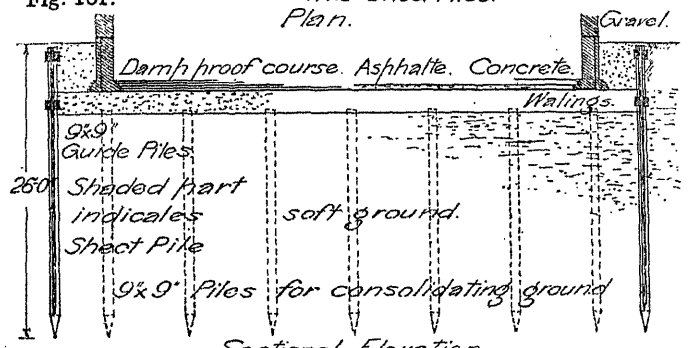


Fig. 162.

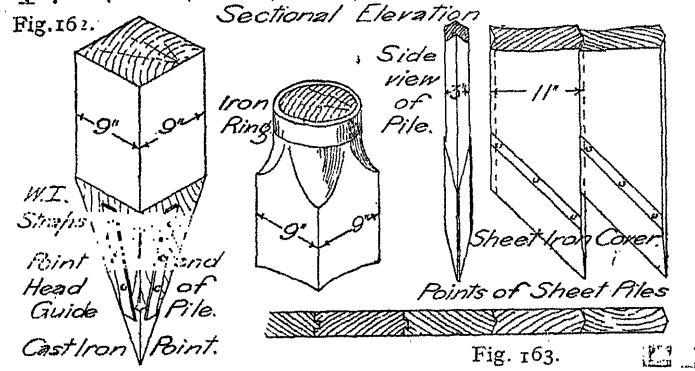


Fig. 163.

Figs. 160—163.

lower extremities, as shown in Fig. 162. The point consists of a pyramidal block of cast iron, about 6 inches in length, and having a base about 4 inches to 5 inches square; this has four mortices, about 2″ × ½″ by about ¾ inch in depth. This is placed on the end of the pile, which has been cut to the form of a truncated pyramid, the iron block completing the latter. It is fixed with four straps of wrought iron about 1 ft. 6 in. in length, with the ends turned to fit in the mortices of the cast-iron points. The straps are fixed to the wood pile with large clout nails, the point being thus fixed, as shown in Fig. 162. The guide piles are driven in to within about 2 feet of the ground; they are connected together by horizontal timbers about 9″ × 6″ bolted to them in pairs, with a space between equal to the thickness of the sheet piles. Two pairs of waling pieces are thus fixed, one at the ground level, and the other near the top of the piles; in the spaces between these the sheet piles are driven, the walings serving to keep them in an upright position. The joints of the sheet piles are prepared in three general ways: square, grooved and tongued, or bird's-mouthed together, the first and last being those most commonly used, the second and third being shown in Fig. 163. The sheeting piles are pointed at their lower ends, in the way shown in Fig. 163, to cause them to draw in one direction; they have a piece of sheet iron nailed over the end to protect the point.

The ground within the enclosure is frequently consolidated by driving in piles, as shown in Fig. 160, the tops of these being covered by a layer of concrete, covering the whole site within the enclosed area.

The enclosed site below the level of the piles is often removed for a few feet in depth, and replaced by a layer of drier material. The foundations are formed by a wide layer of concrete, or by timber platforms.

2. *Soft Soils with Hard Strata beneath.*—Cases of this description are commonly met with where buildings are erected on the banks of rivers; they are usually dealt with in one of two ways—(1) by piling, as shown in Fig. 164, until the pile refuses to be driven ¼ inch at each blow.

(2) By sinking piers down to the firm stratum.

3. *Soils not having a Uniform Resistance.*—Soils of this description, where the ground consists of rock or firm ground in some parts, and in others of a soft soil, require careful treatment to prevent unequal settlement. The best method under these conditions is to cover the whole site with a bed of concrete and a grillage of steel joists, bars or rods embedded in concrete. The soft parts are thus bridged over and a solid, hard platform is obtained over the whole site.

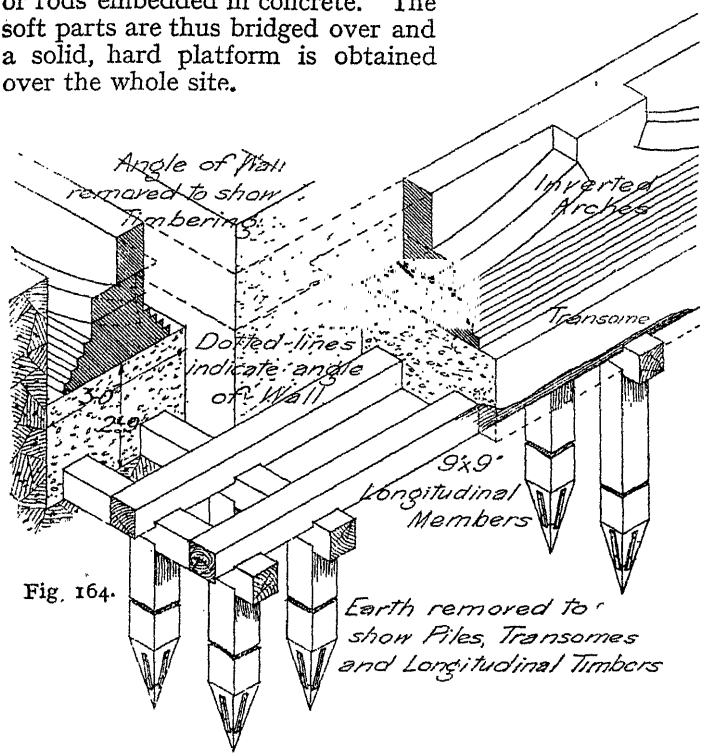


Fig. 164.

Sinking Piers.—Piers of concrete or brick may be taken at intervals through the soft soil down to the hard substratum; these are connected by arches or girders, upon which the superstructure is raised. If the soil is sufficiently firm, timbered excavations are made, and concrete or brick piers may be formed, as shown in Figs. 165 to 167;

but if the soil is waterlogged or in any way insecure, brick cylinders may be sunk, or iron cylinders or caissons, as will be described later, these two cylinders being filled up with concrete forming solid piers.

Concrete Piers.—The following example will illustrate concrete piers. Let a bed of soft soil 30 feet in depth overlie a compact gravel substratum with a safe resistance of 8 tons per super foot. It is required to erect a wall with a load of 8 tons per lineal foot. Let the distance between the centres of adjacent piers be 15 feet, then the total load supported by each pier equals $15 \times 8 = 120$ tons. Let the weight of concrete pier be taken as 40 tons, then the total load on bearing stratum equals $120 + 40 = 160$ tons. The sectional area of concrete pier at base equals $160 \div 8 = 20$ superficial feet. Let the horizontal dimensions of pier to suit brickwork be taken as 5′ 6″ × 4′. A timbered excavation having these internal dimensions is prepared, and then filled solid with concrete, the timbering being removed as the concrete is deposited, or if the ground is uncertain the timbering is frequently left in. Figs. 165 and 166 show the above case.

Brick Piers and Steel Girders.—Let a wall stressed with a load of 8 tons per lineal foot be carried by steel girders supported on brick piers in cement, the centre lines of which are 15 feet apart. Let the safe resistance of brickwork in cement be taken as 10 tons per superficial foot; cement concrete, 1 of Portland cement to 6 parts of ballast, 12 tons per superficial foot; and hard clay as 4 tons per superficial foot. The area of piers = total load on pier ÷ safe resistance of brickwork. Then

$$\text{Area of pier at top course} = \frac{15 \times 8}{10} = 12 \text{ super feet.}$$

Let weight of pier be taken as 20 tons.

$$\text{Area of pier at bottom course} = \frac{\text{total load on footings}}{\text{safe resistance of footings}} = \frac{120 + 20}{10} = 14 \text{ super feet.}$$

The resistance of this cement concrete being greater than the brickwork, it is unnecessary to calculate the footings.

Fig. 165. Fig. 166.

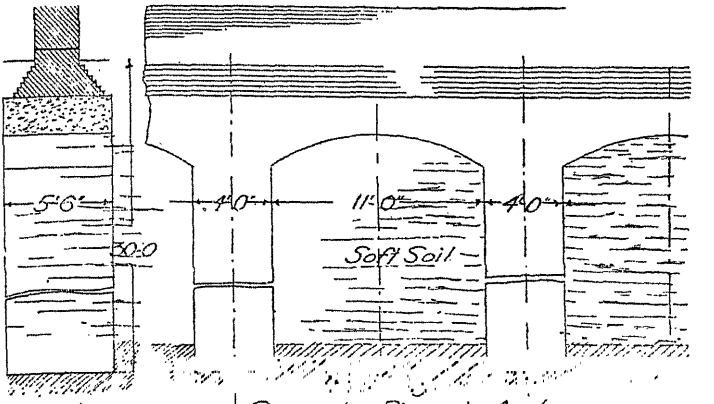


Fig. 167.

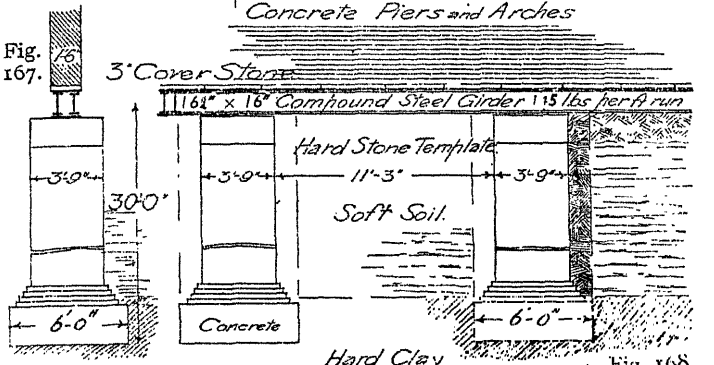


Fig. 168.

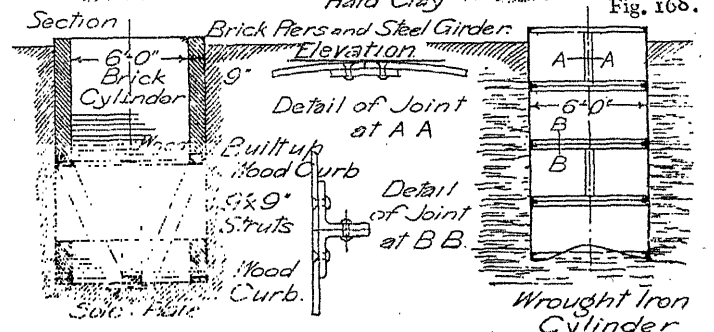


Fig. 169. Fig. 170. Fig. 171.

Figs. 165—171.

Let the weight of the concrete be taken as 5 tons.

$$\text{Area of concrete} = \frac{\text{total load on earth}}{\text{safe resistance of earth}} = \frac{140 + 5}{4} = 36{\cdot}25 \text{ feet}$$

Let the dimensions of the horizontal section of the concrete be taken as 6′ × 6′, and that of the brick pier as 3′ 9″ × 3′ 9″.

The clear span between piers will now equal 11 ft. 3 in., and the total distributed load will equal 90 tons; then from a manufacturer's list a 16$\frac{3}{4}$″ × 16″ compound steel girder, 144·5 lbs. per foot run, will carry a safely distributed load of 93 tons over a 12′ 0″ span. Figs. 167 and 168 illustrate this example.

Sinking Brick Shafts.—Brick shafts are sunk in one of two ways. First, by the method known as underpinning. In this a circular hole is dug in the ground as deep as possible without causing the earth to fall, a circular built-up wood curb is then laid perfectly level at the bottom of the hole, on which the brickwork is raised to the top of the shaft, care being taken to pack the earth tightly behind the brickwork. When this part is completed, a hole is dug in the centre of the shaft as deep as possible, usually from 6 to 8 feet, a wood sole plate is bedded, inclined struts are then inserted, with one end resting on the plate, the other supporting the curb. At the completion of the fixing of these the earth is taken from beneath the curb at all parts to the level of the sole plate. A new curb is now inserted, and the brickwork built up to the underside of the old curb, and the struts removed. This process is repeated till the required depth is obtained, as shown in Fig. 169.

Second method: A wood curb similar to the one in the last method, or an iron curb with a sharp edge, is employed. The curb is laid on the ground, and the brick or stonework raised upon it. The earth inside the curb is removed and on being taken from beneath, the curb with the brickwork sinks, fresh courses of the latter being added as the sinking proceeds. It sometimes happens that the friction on the sides of the brick lining is so great as to prevent the same from sinking; where this occurs, a

second curb is placed inside the first, and a smaller shaft proceeded with in a manner similar to the first.

Iron Cylinder or Caissons.—These are sunk similarly to the brick shaft just described, fresh plates being added as the earth is removed. These being in one mass, as shown in Figs. 170 and 171, there is less likelihood of their going out of the vertical; there is not so much friction on their sides, they therefore sink easier than the above, and are better where there is much water in the soil.

If the above-mentioned shafts have been sunk to the firm strata, they are usually filled with concrete, thus forming a number of solid pillars. These are connected by arches or girders, and upon these the superstructure is raised.

Where soft places such as underground brooks traverse the tracks of walls, it is usual to lay the concrete and bed the footings in the usual manner, and subsequently to build a rough relieving arch through the thickness of the wall over the soft parts, as shown in Figs. 138 and 139.

Soft Soils Subjected to Unequal Pressures.—Where the pressures of a building are concentrated at piers placed at intervals in the length of the wall, there is a danger of unequal settlement and fractures in the unloaded portions of the walls; to counteract this tendency inverted arches, as shown in Figs. 140 and 141, are constructed between the piers, distributing the pressure uniformly over the whole length of the foundations.

Benching.—Where buildings are erected on the side of a hill, and it is not practicable nor economical to excavate the whole site of the building to one level, the ground should be benched—that is, cut into a number of horizontal steps; on these the foundations are laid and the walls raised. The wall, if of brick, should be carried up in cement to the highest level of the foundation; if in masonry, in large blocks of stone, in order to reduce the inequality of settlement due to the varying number of bed joints.

CHAPTER XII

BRICKWORK

CONTINUED FROM THE AUTHOR'S ELEMENTARY COURSE

STABILITY OF WALLS

APART from considerations of defects in the foundations, the stability of walls is affected (*a*) by the unequal distribution or eccentricity of the vertical loads on the walls or piers; (*b*) by side thrusts due to the wind, or untied roof trusses or vaulting.

Walls made of such materials as brick or stone may be supposed, for the purposes of calculation, to be composed of (1) uncemented blocks, or (2) cemented blocks. Walls may be satisfactorily considered as built of uncemented blocks, when the force disturbing its equilibrium acts before the bedding or cementing substance has had time to set; that is, obtains an adhesion and tenacity equal in strength to the materials which are held together, or possesses sufficient of those two resisting powers to withstand the disturbing force. Hence it follows that walls—such as chimney stacks, chimney shafts, buttresses, etc., cases where the failure of one joint would be disastrous, and which cannot be or is not strutted, nor otherwise supported, and will probably have to resist a disturbing element such as wind force before the bedding material has had time to set—should be calculated as if made of uncemented blocks.

Advantage can be taken of the tensile and adhesive resistances of cement mortar, if the requisite conditions with regard to the setting of the mortar are observed. The

tensile resistance of standard cement mortar given in the S.S. is 375 lbs. per square inch after 7 days. For mortar of this quality it would probably be safe to allow up to 90 lbs. per square inch, but where the importance of the work admits, tests should be made with mortar made with the sand available.

Minimum Thickness.—The thickness of walls of dwelling-houses and warehouses has been determined after vast experience, the minimum thickness is given later in this chapter; no advantage would be derived from calculating them.

Fence or boundary walls are usually calculated as built up of a number of cemented blocks—that is, the strength of the mortar is considered, for it may be easily shown that the thickness of the majority of fence walls would be insufficient, were it otherwise, to resist the wind pressures frequently attained.

In order that the maximum resistance may be obtained from a wall or pier, it is essential that the load should be placed axially, every part of the sectional area at right angles to the direction of the load will then be under a uniform pressure. If the load is applied non-axially the pressure over the section will vary. To prevent excessive loading at any point the stress should be determinable at all parts of the section.

Let f = the pressure at any distance y from the centroid
P = total load
p_a = axial pressure
p_e = eccentric pressure
A = total area.

Then for a pier or wall axially loaded

$$p_a = \frac{P}{A}$$

Where the load is applied eccentrically the compressional value of f will be increased on the side of the load by an amount equal to P multiplied by the eccentricity e and divided by the modulus of the section Z (see p. 554). Thus—

$$p_e = \frac{Pe}{Z} = \frac{P\,e\,y}{I}$$

$$f = p_a + p_e = \frac{P}{A} + \frac{P\,e\,y}{I}$$

$$I = r^2 \times A$$

$$\therefore f = \frac{P}{A} + \frac{P\,e\,y}{A r^2}$$

$$= \frac{P}{A}\left(1 + \frac{e\,y}{r^2}\right). \qquad (1)$$

In a symmetrical section such as Fig. 172, $y_1 = y_2$, and the values of f, at the edges, are

$$= \frac{P}{A}\left(1 + \frac{e\,y_2}{r^2}\right)$$

$$= \frac{P}{A}\left(1 - \frac{e\,y_1}{r^2}\right).$$

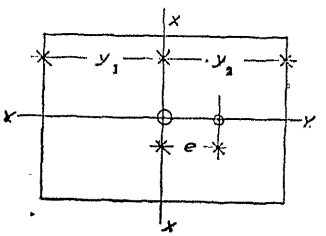


Fig. 172.

If the pier were subjected to a side thrust only, the stresses would be as in the case of a beam, compression on one side of the pillar and tension on the other ; the stresses would be symmetrical about the neutral axes, which would pass through the centroid. These conditions never apply in the case of a pillar or of a pier, as the weight of the structure and the thrust of the load parallel to the axis always result in the compressional stresses on one side of the axis being greater than the tensional stresses on the other.

There are three cases :—

1. Where $\frac{r^2}{e}$ is less than y. The base is partly in compression and partly in tension.

2. Where $\frac{r^2}{e} = y$. The base is wholly in compression, varying from zero on one edge to a maximum on the other.

3. Where $\frac{r^2}{e}$ is greater than y. The base here is wholly in compression, varying from some significant quantity on one edge to a maximum on the other.

Case I. If y_2 represent the distance from xx to the line where the stress changes. Then determine the value of y_2 where $f = o$ (Fig. 173).

$$= o = \frac{P}{A}\left(1 - \frac{ey_2}{r^2}\right)$$

$$= \frac{P}{A} - \frac{P}{A}\frac{e\,y_2}{r^2}$$

$$\therefore y_2 = \frac{r^2}{e}. \qquad (1)$$

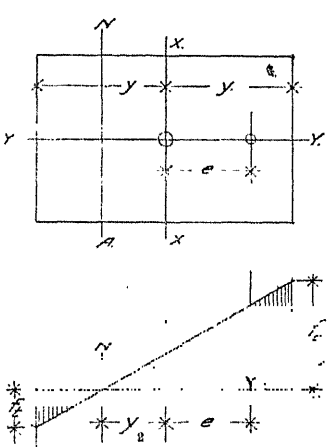


Fig. 173.

Case II. Determine the maximum value of e so that $f = o$ when $y = \frac{d}{2}$ (Fig. 174).

$$f = o = \frac{P}{A} - \frac{Pe\frac{d}{2}}{A r^2}$$

$$e = \frac{2 r^2}{d}$$

$$e = \frac{2 I}{A d} = \frac{2 bd^3}{12 bd^2}$$

$$e = \frac{d}{6}$$

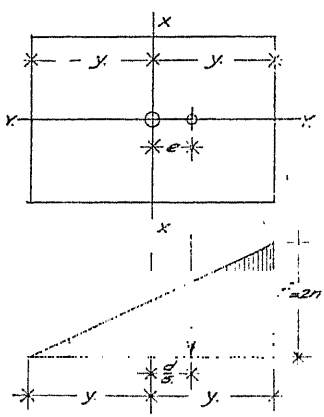


Fig. 174.

If n = average pressure
then $f = 2n$.

Case III. Fig. 175. Here $\frac{r^2}{e}$ is greater than y, that is, the eccentricity is less than $\frac{d}{6}$, the whole of the section will be in compression and f_1 will vary from zero when $e = \frac{d}{6}$ to a maximum where e coincides with the centroid when $f_1 = f$.

In the case of walls having superimposed axial loads to which must be added the weight of the walls or piers, the statement becomes

$$= \frac{Pa}{A} + \frac{P}{A}\left(1 \pm \frac{ey}{r^2}\right).$$

It is evident that Case I. should be employed only where the resistance of the cementing material is reliable, such as

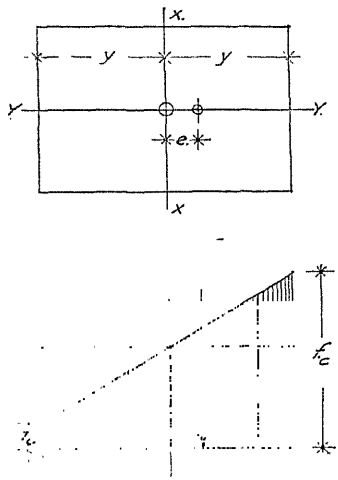


Fig. 175.

when Portland cement mortar is used. Structures built in lime mortar (the resistance of which is always variable and doubtful) should be computed under Cases II. and III.

From Case II. is derived the rule for all structures in brick and stone, *i.e.*, that the resultant of any systems of pressure should fall within the middle third of the structure.

Classification of Failures.—A wall consisting of uncemented blocks (that is, supposing lime mortar to be

used, its tenacity and adhesion being neglected) may fail from the following causes: (*a*) By the overturning of some one or more blocks on their edges; (*b*) by crushing if the pressure be great enough and distributed over too small a surface; (*c*) by the sliding of some one or more of the blocks on their bed joints.

Overturning.—Case (*a*). Let the wind pressure be considered as the external disturbing force. The total wind pressure is the product of the pressure per unit of area, the height and length or area of the exposed face of wall. The moment is the product of the above and its leverage measured from the point of overturning to half the height of the face exposed to the wind.

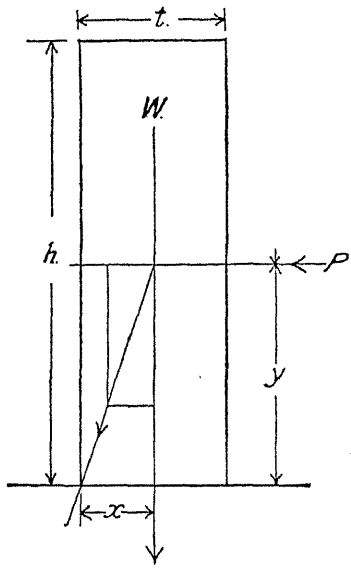


Fig. 176.

The resistance of the wall (assuming that the strength of the materials and the accuracy of the bonding is uniform throughout) will be the product of the weight of the wall and its leverage measured from the perpendicular drawn through its centre of gravity to the point of overturning on the leeward edge of the wall or such other point within the base as may be selected for purposes of safety.

Then let p = wind pressure, P = the resultant force, y = the leverage of the wind, h = the height of the wall, l = its length, t = its thickness, w = weight per cubic foot of wall, x = the leverage of wall. Then for equilibrium (see Fig. 176)—

The moment of the wind pressure = The moment of the mass of a wall

$$= P \times y = W \times x$$

$$= p \times l \times h \times \frac{h}{2} = w \times l \times h \times t \times \frac{t}{2}$$

$$\text{and } t = \sqrt{\frac{ph}{w}}.$$

This gives the expression for the thickness of the wall when on the point of overturning.

The following table gives the velocity and pressure of winds, or the value of P.

TABLE SHOWING THE VELOCITY AND PRESSURE OF WINDS.

Designation.	V = Velocity in miles per hour.	P = Pressure in lbs. per square ft. = ·003 V^2.
Scarcely perceptible...	1	·003
Perceptible	2	·012
Slight breeze...	4	·048
Moderate ,, ...	8	·192
Fresh ,, ...	15	·375
Brisk wind	25	1·875
Strong ,, ...	30	2·700
High ,, ...	40	4·800
Storm...	50	7·500
Violent storm	60	10·800
Hurricane	80	19·200
Violent hurricane	100	30·000
Gust observed in England in 1866...	126	47·628

Let P = pressure of wind in lbs. per square foot against a surface perpendicular to its direction.

If the velocity in miles per hour is known, the value of P may be deduced from the empirical formula $P = \cdot00271\ V^2$, it is sufficiently accurate if $P = \cdot003\ V^2$ be taken.

Wind Pressure.—The Standard Specification 449 requires that: The designs shall allow for wind pressure in any horizontal direction of not less than 15 lbs. per square foot of the upper two-thirds of the vertical projection of the surface of such buildings, with an additional pressure of 10 lbs. per square foot upon all projections above the general roof level. On the sea coast and in

similarly exposed situations a further provision shall be made.

If the vertical projection of a building is less than twice its width, wind pressure may be neglected provided that the building is adequately stiffened by floors and walls.

Note.—The wind loads stipulated in this clause are such as would be regarded as adequate in Great Britain, but may possibly require modification in other countries.

Crushing.—Case (*b*). Consider the walls for safety from crushing, as the equation already stated only satisfies case (*a*), and represents unstable equilibrium or the point of overturning. This condition of safety is usually fulfilled by making the wall of sufficient thickness to meet the requirements of proposition (*b*), which is complied with when the centre of pressure falls sufficiently within the walls for the section of the wall to resist crushing where the stress is at its maximum, which in a wall of ordinary section is at the ground line and on its outer edge.

The expression in order that the whole of the base may be under compression (Case II., p. 294) would be

$$= \sqrt{\frac{3ph}{w}}.$$

If the pathway of the maximum resultant pressure does not intersect the base exactly at the middle third, the values of the extreme stresses on the mortar joint in compression or tension are best determined by the method shown on page dealing with cemented blocks.

Limiting Positions of Centres of Pressure

In order that the whole base may just be under a compressional stress, the method of finding the limiting position of the centre of pressure is shown below for the sections in common use.

I. The solid square or rectangle. From Case II., p. 294,

$$e = \frac{r^2}{y} \qquad r^2 = \frac{d^2}{12}$$

$$e = \frac{d^2 \times 2}{12 \times d} = \frac{d}{6}. \qquad (1)$$

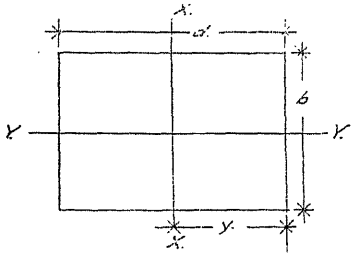


Fig. 177.

II. Solid circle. First determine the I about a polar axis Z at right angles to the plane of the circle.

I of ring about centre = area $\times r^2$
$= 2\pi r, dr \times r^2$

$$\therefore \quad I_z = \int_0^R 2\pi r^3, dr = \frac{\pi R^4}{2} = \frac{\pi D^4}{32}.$$

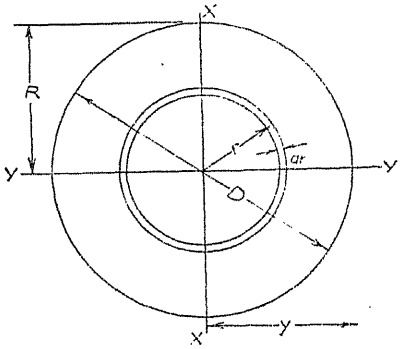


Fig. 178.

Then as the I about any axis equals the sum of the I about any other two axes mutually perpendicular

$$Iz = Ixx + Iyy = 2Ixx = 2Iyy$$

$$\therefore \quad 2Ixx = \frac{\pi D^4}{32} \text{ and } Ixx = \frac{\pi D^4}{64}$$

$$r^2 = \frac{I}{A} = \frac{4\pi D^4}{64\pi D^2} = \frac{D^2}{16}$$

$$\text{then} \quad e = \frac{r^2}{y} = \frac{2 \times D^2}{16 \times D} = \frac{D}{8}. \qquad (2)$$

See Fig. 178.

III. Hollow square or rectangles.

$$e = \frac{r^2}{y}; \; r^2 = \frac{I_2 - I_1}{A_2 - A_1}$$

$$e = \frac{\frac{1}{12}\left(b_2 d_2^3 - b_1 d_1^3\right)}{\frac{d_2}{2}\left(b_2 d_2 - b_1 d_1\right)}$$

$$= \frac{b_2 d_2^3 - b_1 d_1^3}{6 d_2 (b_2 d_2 - b_1 d_1)}$$

See Fig. 179.

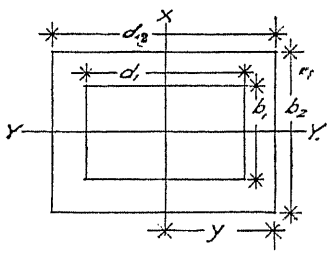


Fig. 179.

IV. The hollow circle. Proceed as with the solid circle. Then

$$Ixx = \frac{\pi}{4}\left(R^4 - R_1^4\right) = \frac{\pi}{64}\left(D^4 - D_1^4\right)$$

$$r^2 = \frac{I}{A} = \frac{\pi 4 (D^4 - D_1^4)}{64\pi (D^2 - D_1^2)} = \frac{D^2 + D_1^2}{16}$$

$$e = \frac{r^2}{y} = \frac{D^2 + D_1^2}{16} \times \frac{2}{D} = \frac{D^2 + D_1^2}{8D}$$

See Fig. 180.

The value of e varies with the thickness of the shell, being a minimum $\frac{D}{8}$ when the figure approaches a solid circle and a maximum $\frac{D}{4}$ with a thin shell.

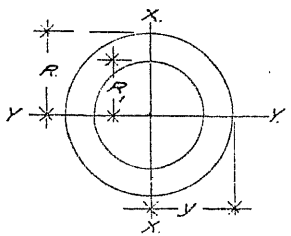


Fig. 180.

Sliding.—The stability of walls with respect to proposition (*c*) or sliding will now be considered. If a rough plane with a body on it be gradually tilted, there is a certain angle to the horizontal when sliding is about to take place. This angle is equal to the angle of repose and is equal to the angle of friction, and the tangent of this angle is known as the coefficient of friction. This angle for a number of materials has been determined experimentally. Whether any particular wall is safe as regards sliding is determined in the following manner (see examples on pp. 333–5). The line of resultant pressure due to the external forces and the weight of the wall is drawn. A normal to the bed joint is drawn to cut this line and the tangent of the angle between the two is found. Now divide the coefficient of friction by the tangent of the angle found and the quotient is the factor of safety against sliding. For safety it is usual not to let this factor of safety be less than 1·25.

MORIN'S TABLES.

Surfaces.	Angle of Repose.	Coefficient of Friction
Dry masonry and brickwork	31° to 35°	0·6 to 0·7
Masonry and brickwork with wet mortar ...	25½°	0·47
Masonry and brickwork with slightly damp mortar...	36½°	0·74
Wood on stone	22°	0·4
Iron on stone	16⅔° to 35°	0·3 to 0·7
Masonry on dry clay	27°	0·51
Masonry on moist clay	18¼°	0·33
Earth on earth	14° to 45°	0·25 to 1·0
Earth on dry sand, clay, and mixed earth ...	21° to 37°	0·38 to 0·75
Earth on damp clay	45°	1·0
Earth on wet clay	17°	0·31
Earth on shingle and gravel	35° to 48°	0·7 to 1·11

EXAMPLE 1.—A chimney stack rises 20 feet clear of roof. It is required to resist a wind pressure of 30 lbs. per foot super. Weight of stack 100 lbs. per cubic foot. Limiting position of centre of pressure $\frac{t}{4}$ from the axis perpendicularly below the CG. Required, the thickness of stack.

In this case from formula for overturning

$$= \sqrt{\frac{2ph}{w}}$$

$$= \sqrt{\frac{2 \times 30 \times 20}{100}}$$

$$= \sqrt{12} = 3{\cdot}46 \text{ feet.}$$

EXAMPLE 2.—What wind pressure would be required for a 14-inch brick wall, 10 feet high, weighing 108 lbs. per cubic foot, to be just on the point of overturning? Neglecting the strength of the mortar.

In this case

$$= \sqrt{\frac{ph}{w}}$$

and $p = \frac{t^2 w}{h}$

$$= \frac{1\frac{1}{6} \times 1\frac{1}{6} \times 108}{10}$$

$$= 14{\cdot}7 \text{ lbs. per super foot.}$$

EXAMPLE 3.—Determine the thickness and the maximum pressure on the base of a brick wall 15 feet high, wind pressure 30 lbs. per super foot. Weight of brickwork 120 lbs. per cubic foot. Neglect the strength of the mortar. In order that the whole of the base may be under compression, the limiting position of the centre of pressure $= \frac{t}{6}$, therefore the statement becomes

$$= \sqrt{\frac{3ph}{w}} = \sqrt{\frac{3 \times 30 \times 15}{120}}$$
$$= \sqrt{11{\cdot}25} = 3{\cdot}35 \text{ feet.}$$

The maximum pressure on the base normal to the bed joint will equal twice the average pressure on the joint.

The weight of the wall

$$= W = 10 \times \quad \times h \times t$$

and pressure per square inch on the base $= \frac{W}{\text{area in inches}}$

and maximum pressure per square inch $= \frac{2W}{\text{area in inches}}$

$$\frac{2 \times 120 \times 1 \times 15 \times 3{\cdot}35}{3{\cdot}35 \times 12 \times 12}$$

$= 25$ lbs. per square inch.

Inclined Thrusts

EXAMPLE 4.—A rectangular wall 10 feet high has a thrust of 500 lbs. per foot run applied at a level of 8 feet above the ground level. The direction of the pressure is 60 degrees to the horizon. Weight of the wall is 100 lbs. per cubic foot. What must be the thickness in order that the base may just be wholly under compression ?

Oblique thrusts should be resolved into their horizontal and vertical components. The horizontal component H will be the disturbing thrust. The vertical component V will act with the wall.

Then $H = 500 \cos 60 = 250$ lbs.
$V = 500 \sin 60 = 433$ lbs.

Consider a length of 1 foot of wall.

Moment due to wall $= W \times \frac{t}{6}$

$$= \frac{100 \times 10 \times t^2}{6} \text{ lbs. ft.}$$

Moment due to $V = 433 \times \frac{2}{3}t = 288{\cdot}7t$ lbs. ft.

Moment due to $H = 250 \times 8 = 2000$ lbs. ft.

Combine these three moments

$$166{\cdot}6t^2 + 288{\cdot}7t - 2000 = 0$$

Solving the quadratic

$$t = \frac{-288{\cdot}7 \pm \sqrt{288{\cdot}7^2 + 4 \times 2000 \times 166{\cdot}6}}{2 \times 166{\cdot}6}$$

$= 2{\cdot}71$ feet.

The thickness $= 2{\cdot}71$ feet.

Then for the maximum pressure on the base

$$P = \left(\frac{\text{Weight of wall + vertical component}}{\text{area of base in inches}}\right)_2$$

$$= \left(\frac{100 \times 10 \times 2{\cdot}71 + 433}{2{\cdot}71 \times 12 \times 12}\right)_2$$

16·1 lbs. per square inch.

Structures built with Cemented Blocks

These cases are considered in a manner very similar to that for beams. They are practically upright cantilevers, having in addition to the transverse load, a load in the direction of the axis consisting of the weight of the structure itself plus any load that may be carried by it. This method of computation may be employed with advantage wherever the tensional and adhesional resistance of the mortar may be relied upon.

These resistances are always doubtful where lime mortars are employed, cement mortars being more uniform in character, give such high results, even when the ultimate resistances have been reduced by a considerable factor of safety, that they may safely be taken into account, if sufficient care is taken to observe all the regulations that produce good results in these mortars.

The following method of calculation is used in the case

of chimneys and shafts built in Portland cement mortar; and for enclosure walls.

Wind pressure is the principal disturbing force acting on walls, and those joints which have to withstand the greatest stress are at the bottom of the wall, where the wind exerts the greatest turning moment.

If the wall be of uniform section throughout, it is quite sufficient if the strength of the lowest joint be calculated; but if the thickness be variable, the lowest joint at each change of section must be taken.

In addition to external forces, there is an internal compressive force due to the weight of the mass of the wall itself. Under certain conditions, however, as in some retaining walls where the joints are not horizontal, this force may cause a portion of the joint to be subjected to a shearing stress, though the strength of mortar and the resistance due to friction are always more than sufficient to withstand that force.

To ascertain the pressure of the wall on the joints, divide the weight of the wall in pounds by the horizontal sectional area in inches, and the result will be the pressure in pounds per square inch—

$$= \frac{W}{A} = \frac{wh}{144}.$$

For the pressure per unit of area due to the wind

$$\left.\begin{matrix}\text{The moment of the}\\ \text{wind pressure}\end{matrix}\right\} = \left\{\begin{matrix}\text{The moment of the}\\ \text{resistance of wall}\end{matrix}\right.$$

$$\text{that is,} \qquad M = \frac{fI}{y}$$

where I = moment of inertia of wall, and y = distance of the windward or leeward edge from the neutral axis of wall.

Then from the above statement, the intensity of pressure f at any distance y from the neutral axis may be expressed as follows :—

$$M_R = M_P$$

$$\frac{fI}{y} = \frac{p \,.\, l \,.\, h \,.\, h}{2} \text{ and } f = \frac{p \,.\, l \,.\, h^2 \,.\, y}{2\,I}$$

$$\text{but } I = \frac{bd^3}{12} \text{ and } y = \frac{d}{2}$$

$$\therefore \quad = 3\,\frac{plh^2}{bd^2}.$$

$$\text{Then total compression} = f_c + \frac{W}{A}.$$

$$\text{,, ,, tension} = f_t - \frac{W}{A}.$$

EXAMPLE.—Determine the pressure per square inch on the windward and leeward edges of a long enclosure wall, height 12 feet, thickness 1 ft. 6 in., weighing 100 lbs. per cubic foot. Wind pressure 30 lbs. per super foot. The wall to be built in cement mortar in the proportion of 1 part Portland cement to 3 parts washed river sand.

Referring to the formula given $d = t$ and $b = l$, let $l = 1$ foot. This being a rectangular wall in section w per inch on base $= \frac{wh}{144}$.

$$M_R = M_P$$

$$\frac{fI}{y} = \frac{p \,.\, l \,.\, h^2}{2} \text{ and } f = \frac{3\,plh^2}{bd^2}$$

$$\therefore f = \frac{3 \times 30 \times 12 \times 144 \times 144}{144 \times 12 \times 18 \times 18} = 40 \text{ lbs. sq. in.}$$

$$\frac{W}{A} = \frac{100 \times 12}{144} = 8{\cdot}33 \text{ lbs. sq. in.}$$

$ft = 40 - 8{\cdot}33 = 31{\cdot}67$ lbs. per square inch in windward edge of wall

$fc = 40 + 8{\cdot}33 = 48{\cdot}33$ lbs. per square inch on leeward edge of wall.

Taking the values given in the Standard Specification for cement mortar of 375 lbs. per square inch

$$\text{The factor of safety on the leeward edge} = \frac{375}{31{\cdot}67} = 11{\cdot}9$$

Taking the value of brickwork for crushing from the class 5,000—7,500 with 1—3 cement mortar from the S.S. 449, the maximum permissible pressure equals 16 tons per square foot, or 249 lbs. per square inch. Thus there is an ample margin of safety.

EXAMPLE.—Given a long boundary wall 8 feet high, 9 inches thick, having buttresses 1 ft. 6 in. wide, 4½ inches projection on both sides of the wall at 12 feet centres. Weight of brickwork 120 lbs. per cubic foot. Wind pressure 30 lbs. per superficial foot. Determine the maximum

intensity of pressure along the extreme edges of the buttresses.

Weight of wall per square inch on base $= \frac{8 \times 120}{144} = 6{\cdot}6$ lbs. per square inch

Moment of wind pressure on one bay $= \frac{Ph}{2} = (p \times l \times h) \times \frac{h}{2}$

$= 2{,}880 \times 48$

$= 138{,}440$ inch lbs.

Moment of resistance $= \frac{f(I_1 + I_2)}{y}$

$= \frac{f}{6}\left(\frac{l_1 t_1^3 + l_2 t_2^3}{d_1}\right)$

$= \frac{f}{6}\left(\frac{18 \times 18^3 + 126 \times 9^3}{18}\right)$

$= f\,\frac{196{,}830}{108} = 1{,}822{\cdot}5f.$

$M_R = M_P$

$1{,}822{\cdot}5 = 138{,}440$

$f = 75{\cdot}9$

$\frac{W}{A} = 6{\cdot}6$ lbs.

$f_t = 75{\cdot}9 - 6{\cdot}6 = 69{\cdot}3$ lbs. per square inch.

$f_c = 75{\cdot}9 + 6{\cdot}6 = 82{\cdot}5$ " " " "

For the tensional and adhesional values given the above result shows an ample margin of safety. The margin in compression = 82·5 is safe. As a wind pressure of 30 lbs. per square foot very seldom occurs, these stresses may be considered satisfactory.

Thicknesses of Walls.—The following are the Byelaws made by the London County Council in pursuance of the London Building Act (Amendment) Act, 1935.

PART IV.—WALLS AND PIERS

Section 1.—*General Requirements.*

39.—Every building shall be enclosed with walls. Provided that openings may be made in such walls subject to the following conditions—(1) That the total elevational area of openings in any such wall above the soffit of the first floor do not exceed one-half the elevational area of such wall measured from the soffit of the first

floor of the building to the roof; (2) that the total elevational area of openings in any storey-height of such wall above the soffit of the first floor of the building do not exceed two-thirds of the total area of such wall within such storey-height; (3) that the total width of openings of any level above the soffit of the first floor do not exceed three-quarters of the total length of the wall at that level. For the purposes of this byelaw, the expression "walls" shall be deemed to include piers and for the purpose of this byelaw and of byelaws 43 and 51 (*g*) any glazing or glass in the thickness of such walls shall be deemed to be an opening.

40.—Every wall or pier of a building shall be constructed of bricks or blocks laid in horizontal courses properly bonded, bedded and jointed with mortar or of plain concrete or of reinforced concrete or (except in the case of party walls) of such materials in combination with metal framework.

Where any walls of a building meet, or where such walls meet piers, they shall be properly bonded or otherwise securely and permanently bound together.

41.—No hollow bricks or hollow blocks shall be used in the construction of a wall or pier of a building (other than a non-load bearing partition wall) unless evidence has been produced to the satisfaction of the Council showing that such wall or pier will be equal as regards fire-resistance to that of a wall or pier constructed of solid bricks or solid blocks or of plain or reinforced concrete in accordance with the requirements of these byelaws.

42.—No timber or other combustible material (other than the ends of beams, joists, purlins and rafters, the horns of door frames and of window frames, fixing blocks and plugs and pole plates bearing rafters and supporting no walling other than wind-pinning) shall be built into the required thickness of a wall or pier and when the end of a beam, joist, purlin, rafter or other timber is built into the required thickness of a party wall, it shall not extend beyond the middle of the wall and shall be encased in brick or other solid incombustible material not less than 4 inches in thickness.

43.—No external wall, party wall or buttressing wall constructed of bricks or blocks or plain concrete shall be of less thickness in any part than $8\frac{1}{2}$ inches, exclusive of plastering, rendering, rough cast or other applied covering. No reinforced concrete external wall or reinforced concrete part or panel of an external wall shall be of less thickness in any part than 4 inches exclusive of plastering, rendering, rough cast or other applied covering. No reinforced concrete party wall shall be of less thickness in any part than 8 inches, and no such party wall forming part of a building of the warehouse class, such building being constructed otherwise than as a reinforced concrete building, shall be of less thickness in any part than 13 inches exclusive of plastering, rendering, roughcast or other applied covering in each case.

Provided that:—

(i) a building of not more than one storey in height, not being a dwelling house, and the width of which (measured in the direction

of the span of the roof) does not exceed 30 feet and the height of the walls of which does not exceed 10 feet; or

(ii) an erection situated above the level of the roof of a building and intended for the protection of a tank or motor or for a like purpose, and not intended for or adapted to use for habitable purposes or as a work room, such erection being adequately supported to the satisfaction of the district surveyor, and not exceeding 10 feet in either length or width and not exceeding 8 feet in height measured from the level of the roof of the building to the top of the walls of such erection; may be enclosed with external walls constructed of bricks or blocks and not less than 4 inches thick subject to the following conditions:—

(*a*) That any such wall be bonded into piers of the size required by calculations based on the loads and stresses specified in these byelaws, but not less than $8\frac{1}{2}$ inches square in horizontal section.

(*b*) That such pier be provided at each end of such external wall.

(*c*) That in the case of (i) further similar piers be provided if any such wall exceeds 10 feet in length, as may be necessary so to divide the wall that the length of each portion of such wall shall not exceed 10 feet measured in the clear between such piers.

(*d*) That all bedding and jointing be in cement mortar.

(*e*) That the roof be so constructed that the walls are not subject to any thrust therefrom.

(*f*) That no load other than a distributed load of the roof be borne by the walls.

Every party wall which exceeds 30 feet in height shall have a thickness of solid material in every part thereof of not less than 13 inches. Provided that this requirement shall not apply to a wall which does not serve to enclose a building of the warehouse class, and (*a*) which does not exceed 40 feet in height; or (*b*) which does not exceed 35 feet in length and 50 feet in height.

Where any part of a party wall serves to divide basements, such part shall not have a less thickness of solid material than 13 inches. Provided that this requirement shall not apply to a party wall not forming part of a building of the warehouse class and not exceeding 25 feet in height and not exceeding 30 feet in length.

44.—Notwithstanding anything in these byelaws, every wall and pier of a building constructed of bricks or blocks or of plain concrete shall have a thickness at every level equal to not less than one-sixtieth of the height to the top of such wall measured from that level, and every party wall and every pier combined therewith of such construction shall have a thickness of solid material at every level equal to not less than one-fortieth of the height to the top of such wall measured from that level.

Provided that if a wall of a building be so constructed that any part thereof does not in itself sustain nor aid in sustaining any of the loads on the rest of such wall and the rest of such wall be of sufficient strength, stability and stiffness to resist all the loads on the whole of such wall, such part of such wall, being properly bonded,

or otherwise adequately joined to the rest of such wall, may, for the purpose of determining the thickness of such part be deemed to be a separate wall or panel or separate walls or panels.

45.—Cavity walls shall be constructed of bricks or blocks properly bedded and jointed and shall comprise two leaves, each not less than 4 inches thick and an intervening cavity not less than 2 inches and not more than 6 inches wide. The two leaves shall be united by iron ties so shaped as not to transmit moisture across the cavity and not less than $\frac{3}{4}$ inch by $\frac{1}{16}$ inch in cross section, well galvanized or otherwise protected from corrosion or by ties of such other material and cross section as may be approved as of like suitability by the district surveyor. Such ties shall be evenly distributed and so disposed that where the cavity is not more than 3 inches wide, there shall be not less than two ties to every superficial yard and they shall be not more than 3 feet apart horizontally and there shall also be one to every [illegible] the sides of all openings. Where the cavity is 6 incl[illegible] of wall ties employed shall be twice that required when the cavity is not more than 3 inches wide. Where the cavity is more than 3 inches and less than 6 inches wide, the number of ties employed and their spacing shall be correspondingly proportionate to the numbers and spacing required where the cavity is not more than 3 inches wide and the numbers and spacing where the cavity is 6 inches wide. The cavity of all cavity walls shall be free from mortar droppings and debris. Where woodwork extends into or across such cavity, sheet lead or other permanent impervious material shall be so built into such wall as to protect such woodwork from moisture descending the cavity and so as not to convey such moisture across the cavity.

46.—Every building shall be so constructed as to ensure that it will not be affected adversely by moisture from adjoining earth.

47.—In every building, the top of every wall constructed of bricks or blocks and not otherwise protected from the weather shall be so protected by a coping of bricks set on edge in cement mortar on a creasing of two courses of slates or dense tiles set in cement mortar to break joint or on some equally suitable damp-proof course, or shall be so protected by some equally suitable coping. All such coping and creasing shall be bedded and secured to the satisfaction of the district surveyor.

48.—No earth, concrete, brickwork, stonework or other material supporting or aiding in the support of any superstructure shall be disturbed within two clear days of the district surveyor having received notice in writing giving particulars of the nature of the work and the date of its commencement.

49.—A wall shall not be thickened except after two clear days' notice being served on the district surveyor of the intention to thicken, and the thickening shall be executed to the satisfaction of the district surveyor and such wall so thickened shall be of the required thickness.

50.—A wall or pier shall not be deemed to sustain and transmit

all the dead and superimposed loading as required by byelaw 2 unless such wall or pier is in comformity with either

(*a*) the prescribed conditions, dimensions and other requirements set out in Section 2 of this part of these byelaws ; or

(*b*) the limits of stress and other requirements set out in Section 3 of this part of these byelaws, and in Parts V and VI of these byelaws.

Section 2.—Rules for The Determination of the Thickness of Walls when such Thicknesses are not Determined by Calculation of Stresses under Section 3

51.—This section shall apply only to walls complying with the following conditions (herein called " the prescribed conditions ").

(*a*) That the wall is constructed of bricks or blocks laid in horizontal courses and properly bonded, bedded and jointed with mortar or plain concrete not inferior to that designated IV in byelaw 14.

(*b*) Subject to the provisions of paragraph (*g*) following, that if any part of the wall overhang any part beneath it, the projection of such overhang is in addition to the required thickness, and the amount of such projection does not exceed one-third of such required thickness, that the projecting portion is corbelled out or otherwise supported to the satisfaction of the district surveyor, and except where a part of a wall is, by virtue of the proviso to byelaw 44 deemed to be a separate wall, that no diminution in the thickness of the wall is made on the side of the wall opposite to the projecting portion where such projecting portion occurs in the wall, and that where such projecting portion is constructed as a cavity wall, the projection of such overhang does not exceed one-third the thickness of the leaf so overhanging unless such portion, otherwise constructed as a cavity wall is solid at the bottom to a height equal to its overall thickness.

(*c*) That the wall does not aid in supporting more than one storey in the roof.

(*d*) That for the purposes of byelaws 54 and 55 the storey-heights of all walls are deemed to be determined by their bases and the lateral supports afforded by the floors (other than the lowest floor) and the roof, and that such floors and roof (including the floor of any storey in the roof) so bear on, or are otherwise so attached to such walls as to afford adequate lateral support, to the satisfaction of the district surveyor, and that where such floors and roof on the opposite sides of the same wall are at different levels, the storey-heights of such wall are determined by the lateral supports on one side only. Provided that where the lowest floor is attached to the walls in the manner aforesaid at a level not less than 8 feet above the bases of the walls, the lateral support of such floor may be regarded as determining storey-heights.

(*e*) That the wall is not subject to loads other than distributed loads. Provided that joists which are set at distances apart not

exceeding 42 inches measured centre to centre and which bear directly on the wall or, in the case of timber floors, are properly trimmed into trimming joists so bearing on the wall, shall be deemed to impose a distributed load.

(*f*) That the difference between the levels of the surface of the concrete or ground on one side of any such wall and that of the concrete or ground on the other side does not exceed three times the thickness of the wall measured at the higher of the two levels.

(*g*) That the total elevational area of the openings and of recesses formed in any storey-height in the required thickness of any cross wall, party wall or other internal wall which is a buttressing wall and in any external wall in any storey-height above the soffit of the first floor does not exceed one-half the elevational area of such wall in such storey-height and that the superstructure above any such opening or recess is borne on a suitable arch, beam or lintel of incombustible material adequately supported at each end on a pier or wall of not less width measured on the face that one-sixth that of such opening or recess, except where such recess does not exceed 5 inches in depth and the superstructure is borne on suitable corbelling to the satisfaction of the district surveyor. That unless the district surveyor is satisfied that such opening or recess will not prejudice the stability of the building or any part thereof, no opening or recess (other than in an external wall below the soffit of the first floor) is made in any buttressing wall within a distance measured horizontally in the clear equal to five times the thickness of the wall to which such buttressing wall is bonded. The expression " Recess " shall be deemed to include all chases and other reductions in the required thickness of the wall.

52.—Walls shall be deemed to be divided into distinct lengths by buttressing walls or buttressing piers so bonded or tied thereto as to afford adequate lateral support throughout the height of the wall so deemed to be divided, up to the underside of the floor joists or (if there be no floor joists) to the soffit of the floor of the topmost storey, or (if there be not more than one storey), to the top of such wall. The clear dimensions between such buttressing walls or buttressing piers shall be deemed to be the measure of such lengths.

53.—Every external wall or party wall shall be of the thickness required by byelaw 54 or 55. Every wall buttressing (but not being) an external wall or party wall, shall be not less than two-thirds of the required thickness for an external or party wall of the same height and length and belonging to the same class of buildings. Every partition wall and every wall buttressing a partition wall shall have a thickness of not less than one-half the required thickness for an external or party wall of the same height and twice the length and belonging to the same class of buildings. Wherever an internal wall becomes in any part an external wall, that part of such wall and the part beneath it shall be of the thickness hereinbefore required for an external wall of the same height and length and belonging to the same class of buildings.

Provided that :—

(*a*) A non-load-bearing partition wall adequately restrained laterally on all four edges and otherwise buttressed or restrained as may be necessary to the satisfaction of the district surveyor, may be of a less thickness subject to such partition wall being of such dimensions that when three times its height is added to its length, the total does not exceed two hundred times its thickness. Notwithstanding anything in this Section of these byelaws the bricks or blocks in such non-load-bearing partition wall may be bedded and jointed in calcium sulphate plaster.

(*b*) An external wall not exceeding 25 feet in height and not exceeding 30 feet in length, not being a panel or part of a wall as provided in byelaw 56 and being adequately supported laterally at each end, may be constructed as a cavity wall in accordance with byelaw 45. In determining the thickness of such wall in relation to storey-height for the purpose of byelaw 54 or 55 (as the case may be) the thickness shall be deemed to be the sum of the thicknesses of the two leaves.

(*c*) Where an increase of thickness is, by any byelaw in this Section, required in the case of a wall exceeding 60 feet in height and 45 feet in length or in the case of a storey-height exceeding sixteen times or fourteen times (as the case may be) the thickness prescribed for walls, or in the case of a wall below such storey-height, the increased thickness may be confined to piers properly distributed of which the collective widths amount to not less than one-fourth part of the length of the wall.

54.—External and party walls shall, in a building other than a public building or a building of the warehouse class, be of not less thickness than the thickness hereinafter specified in each case, that is to say :—

(*a*) Where the wall does not exceed 12 feet in height it shall, whatever its length may be, be 8½ inches thick.

(*b*) Where the wall exceeds 12 feet but does not exceed 25 feet in height its thickness shall be as follows :—

If the wall does not exceed 30 feet in length it shall be 8½ inches thick for its whole height.

If the wall exceeds 30 feet in length it shall be 13 inches thick throughout the lowermost storey-height and 8½ inches thick for the rest of its height.

(*c*) Where the wall exceeds 25 feet but does not exceed 30 feet in height its thickness shall be as follows :—

If the wall does not exceed 20 feet in length it shall be 8½ inches thick for its whole height.

If the wall exceeds 20 feet but does not exceed 30 feet in length it shall be 13 inches thick throughout the lowermost storey-height and 8½ inches thick for the rest of its height.

If the wall exceeds 30 feet in length it shall be 13 inches thick throughout the lowermost two storey-heights and 8½ inches thick for the rest of its height.

(*d*) Where the wall exceeds 30 feet but does not exceed 40 feet in height its thickness shall be as follows :—

If the wall does not exceed 35 feet in length it shall be 13 inches thick throughout all storey-heights except the topmost storey-height.

If the wall exceeds 35 feet in length it shall be 17½ inches thick for the lowermost storey-height, 13 inches thick for the rest of its storey-heights below the topmost storey-height, and 8½ inches thick for the topmost storey-height.

(*e*) Where the wall exceeds 40 feet but does not exceed 50 feet in height its thickness shall be as follows :—

If the wall does not exceed 35 feet in length it shall be 17½ inches thick for the lowermost storey-height, 13 inches thick for the rest of its height below the topmost storey-height, and 8½ inches thick for the topmost storey-height.

If the wall exceeds 35 feet but does not exceed 45 feet in length it shall be 17½ inches thick for the lowermost two storey-heights and 13 inches thick for the rest of its height.

If the wall exceeds 45 feet in length it shall be 21½ inches thick for the height of the lowermost storey-height, 17½ inches thick for the next storey-height and 13 inches thick for the rest of its height.

(*f*) Where the wall exceeds 50 feet but does not exceed 60 feet in height its thickness shall be as follows :—

If the wall does not exceed 45 feet in length it shall be 17½ inches thick for the lowermost two storey-heights, and 13 inches thick for the rest of its height.

If the wall exceeds 45 feet in length it shall be 21½ inches thick for the lowermost storey-height, 17½ inches thick for the next two storey-heights, and 13 inches thick for the rest of its height.

(*g*) Where the wall exceeds 60 feet but does not exceed 70 feet in height its thickness shall be as follows :—

If the wall does not exceed 45 feet in length it shall be 21½ inches thick for the lowermost storey-height, 17½ inches thick for the next two storey-heights, and 13 inches thick for the rest of its height.

If the wall exceeds 45 feet in length it shall (subject to proviso (*c*) of byelaw 53 respecting distribution in piers) be increased in thickness except in the uppermost two storey-heights, by 4½ inches.

(*h*) Where the wall exceeds 70 feet but does not exceed 80 feet in height its thickness shall be as follows :—

If the wall does not exceed 45 feet in length it shall be 21½ inches thick for the lowermost storey-height, 17½ inches thick for the next three storey-heights, and 13 inches thick for the rest of its height.

If the wall exceeds 45 feet in length it shall (subject to proviso (*c*) of byelaw 53 respecting distribution in piers) be increased in thickness, except in the uppermost two storey-heights, by 4½ inches.

(*i*) Where the wall exceeds 80 feet but does not exceed 90 feet in height its thickness shall be as follows :—

If the wall does not exceed 45 feet in length it shall be 26 inches thick for the lowermost storey-height, 21½ inches thick for the next storey-height, 17½ inches thick for the next three storey-heights, and 13 inches thick for the rest of its height.

If the wall exceeds 45 feet in length, it shall (subject to proviso (*c*) of byelaw 53 respecting distribution in piers) be increased in thickness, except in the uppermost two storey-heights, by 4½ inches.

(*j*) Where the wall exceeds 90 feet but does not exceed 100 feet in height its thickness shall be as follows :—

If the wall does not exceed 45 feet in length it shall be 26 inches thick for the lowermost storey-height, 21½ inches thick for the next two storey-heights, 17½ inches thick for the next three storey-heights, and 13 inches thick for the rest of its height.

If the wall exceeds 45 feet in length it shall (subject to proviso (*c*) of byelaw 53 respecting distribution in piers) be increased in thickness, except in the uppermost two storey-heights, by 4½ inches.

(*k*) Where the wall exceeds 100 feet but does not exceed 120 feet in height its thickness shall be as follows :—

If the wall does not exceed 45 feet in length it shall be 30 inches thick for the lowermost storey-height, 26 inches thick for the next two storey-heights, 21½ inches thick for the next two storey-heights, 17½ inches thick for the next three storey-heights, and 13 inches thick for the rest of its height.

If the wall exceeds 45 feet in length it shall (subject to proviso (*c*) of byelaw 35 respecting distribution in piers) be increased in thickness except in the uppermost two storey-heights, by 4½ inches.

If, in any storey-height of an external or party wall of a building, other than a public building or a building of the warehouse class, the thickness of the wall as determined by the foregoing provisions of these byelaws is less than one-sixteenth part of the storey-height, the thickness of the wall shall be increased to one-sixteenth part of the storey-height, and the thickness of such wall below that storey-height shall be increased to a like extent.

55.—External and party walls shall, in a building of the warehouse class, be at the base not less thick than hereinafter specified in each case. The thickness of the wall at the top and for 16 feet below the top shall (except in the case of the topmost storey-height of a wall where such wall does not exceed 30 feet in height) be not less than 13 inches, and the intermediate parts of the wall between the base and 16 feet below the top shall not be of less thickness than if the wall were to be built solid throughout the space between straight lines drawn on each side of the wall and joining the thickness at the base to the thickness at 16 feet below the top, or, where hereinafter specified, not less than 4½ inches in excess of such thickness.

(*a*) Where the wall does not exceed 25 feet in height it shall, whatever its length may be, be 13 inches thick at the base.

(*b*) Where the wall exceeds 25 feet but does not exceed 30 feet in height it shall be of the thickness following :—

If the wall does not exceed 45 feet in length it shall be 13 inches thick at the base.

If the wall exceeds 45 feet in length it shall be 17½ inches thick at the base.

(*c*) Where the wall exceeds 30 feet but does not exceed 40 feet in height it shall be of the thickness following :—

If the wall does not exceed 35 feet in length it shall be 13 inches thick at the base.

If the wall exceeds 35 feet but does not exceed 45 feet in length it shall be 17½ inches thick at the base.

If the wall exceeds 45 feet in length it shall be 21½ inches thick at the base.

(*d*) Where the wall exceeds 40 feet but does not exceed 50 feet in height it shall be of the thickness following :—

If the wall does not exceed 30 feet in length it shall be 17½ inches thick at the base.

If the wall exceeds 30 feet but does not exceed 45 feet in length it shall be 21½ inches thick at the base.

If the wall exceeds 45 feet in length it shall be 26 inches thick at the base.

(*e*) Where the wall exceeds 50 feet but does not exceed 60 feet in height it shall be of the thickness following :—

If the wall does not exceed 45 feet in length it shall be 21½ inches thick at the base.

If the wall exceeds 45 feet in length it shall be 26 inches thick at the base.

(*f*) Where the wall exceeds 60 feet but does not exceed 70 feet in height it shall be of the thickness following :—

If the wall does not exceed 45 feet in length it shall be 21½ inches thick at the base.

If the wall exceeds 45 feet in length it shall (subject to proviso (*c*) of byelaw 53 respecting distribution in piers) be increased in thickness from the base up to within 16 feet from the top of the wall by 4½ inches.

(*g*) Where the wall exceeds 70 feet but does not exceed 80 feet in height it shall be of the thickness following :—

If the wall does not exceed 45 feet in length it shall be 21½ inches thick at the base.

If the wall exceeds 45 feet in length it shall (subject to proviso (*c*) of byelaw 53 respecting distribution in piers) be increased in thickness from the base up to within 16 feet from the top of the wall by 4½ inches.

(*h*) Where the wall exceeds 80 feet but does not exceed 90 feet in height it shall be of the thickness following :—

If the wall does not exceed 45 feet in length it shall be 26 inches thick at the base.

If the wall exceeds 45 feet in length it shall (subject to proviso (*c*) of byelaw 53 respecting distribution in piers) be increased in thick-

ness from the base up to within 16 feet from the top of the wall by 4½ inches.

(*i*) Where the wall exceeds 90 feet but does not exceed 100 feet in height it shall be of the thickness following :—

If the wall does not exceed 45 feet in length it shall be 26 inches thick at the base.

If the wall exceeds 45 feet in length it shall (subject to proviso (*c*) of byelaw 53 respecting distribution in piers) be increased in thickness from the base up to within 16 feet from the top of the wall by 4½ inches.

(*j*) Where the wall exceeds 100 feet but does not exceed 120 feet in height it shall be of the thickness following :—

If the wall does not exceed 45 feet in length it shall be 31 inches thick at the base.

If the wall exceeds 45 feet in length it shall (subject to proviso (*c*) of byelaw 53 respecting distribution in piers) be increased in thickness from the base up to within 16 feet from the top of the wall by 4½ inches.

If, in any storey-height of an external or party wall of a building of the warehouse class, the thickness of the wall as determined by the foregoing provisions of these byelaws is less than one-fourteenth part of the storey-height, the thickness of the wall shall be increased to one-fourteenth part of the storey-height and the thickness of such wall below that storey-height shall be increased to a like extent.

56.—If a part or panel of a wall, by virtue of the proviso to byelaw 44 be deemed to be a separate wall for the purpose of determining the thickness, the height of such part or panel shall not exceed 25 feet and the height or the length thereof (whichever is the less) shall not exceed eighteen times the thickness exclusive of rendering, plaster, stone or other slab facing or other decorative finish.

If such part or panel be constructed as a cavity wall, the height or length thereof (whichever is the less) shall not exceed 13 feet and the area thereof shall not exceed 200 square feet.

57.—Underpinning shall be carried out to the satisfaction of the district surveyor to the full thickness of the old work or to the required thickness, whichever be the greater.

Section 3.—*Rules as to the Permissible Stresses in Walls and Piers for the purpose of Calculation where the Thicknesses are not Determined under Section* 2

58.—If in a storey-height, part of a wall is borne by a pier or a pier is borne by part of a wall, such pier together with the part of the wall of the storey-height directly above or below shall be deemed to be a pier extending throughout such storey-height.

Where a wall and a pier are horizontally in structural combination and the pier projects from one or both faces of the wall, if such projection from one face of the wall does not exceed one-quarter the

thickness of such wall, or if the sum of two projections from the two faces does not exceed one-third the thickness of the wall, such combination shall, for the purposes of this Section of these byelaws be deemed to be a wall. If such projection or the sum of such projections exceed one-quarter or one-third the thickness of such wall respectively, and if such additional thickness or width constitutes part of the required thickness, such combination shall be deemed to be a pier the thickness or width of which shall be measured from the face of the projection on the one side of the wall to the face of the other side of the wall, or if such pier project also from the other side of the wall, to the face of the projection on the other side of the wall.

59.—For the purpose of byelaw 60 the slenderness ratio of any storey-height of a wall or pier constructed of bricks or blocks or of plain concrete shall be the ratio of the effective height to the horizontal dimension lying in the direction of the lateral support determining such storey-height.

For the purpose of this byelaw, the effective height shall be :—

In the case of a storey-height of a wall with lateral support at the top thereof, [illegible] times [illegible].

In the case of a storey-height of a wall with lateral support at the top thereof, three-quarters of such storey-height.

In the case of a storey-height of a pier without lateral support at the top thereof, twice such storey height.

In the case of a storey-height of a pier with lateral support at the top, the actual storey-height.

60.—If any storey-height of a wall or pier not having a slenderness ratio exceeding 6, be constructed of bricks or blocks, the total compressive stress due to the vertical load, horizontal pressure and to any other forces shall not exceed the maximum pressure indicated in Table VII (following) in respect of the designation of the bricks or blocks employed as regards strength or of the proportions of the mortar employed, whichever is the weaker. If such storey-height be constructed of plain concrete, such compressive stress shall not exceed the maximum pressure indicated in Table VIII (following).

If, in any case, the designation of the materials are not determined for the purposes of Tables VII or VIII (following), the maximum permissible pressure shall be as approved by the district surveyor.

In all cases of thickening of walls, or the combination of new work with old in any other manner, the permissible stresses shall be as approved by the district surveyor.

If in any wall or pier and in the same storey-height materials differing in designation be employed, the weakest shall be deemed to be employed.

If such wall or pier or any storey-height thereof have a slenderness ratio of 12 such total compressive stress shall not exceed 40 per cent. of the corresponding maximum pressure indicated in Table VII or Table VIII (following) as the case may be. If in such wall or pier or in any storey-height thereof, the greater slenderness

TABLE VII.—MAXIMUM PERMISSIBLE PRESSURES ON WALLS AND PIERS OF BRICKS AND BLOCKS

Designation of bricks or blocks as regards strength (as specified in byelaw 19).	Proportion of Mixture of Mortar (in Volumes).			Maximum pressure in tons per square foot.
	Cement.	Lime.	Sand.	
Special	1	—	2	$\frac{1}{500}$ of the number of lb./sq. inch of the resistance of the bricks or blocks to crushing + 10, but not exceeding 40.
1st	1	—	$2\frac{1}{2}$	.. 30
2nd	1	—	$2\frac{1}{2}$	.. 23
3rd	1	—	3	.. 16
4th	1	—	3	.. $13\frac{1}{2}$
5th	1	—	4	.. 11
5th	1	1	6	.. 10
6th	1	—	4	.. 8
6th	1	1	6	.. 7
6th	1	2	9	.. 6
6th	1	3	12	.. $5\frac{1}{2}$
6th	1	4	15	.. 5
6th	1	5	18	.. $4\frac{1}{2}$
6th	—	1	3	.. 4

TABLE VIII.—MAXIMUM PERMISSIBLE PRESSURES ON WALLS AND PIERS OF PLAIN CONCRETE

Designation of concrete as regards strength (as specified in byelaw 14).	Cubic feet of aggregate per 112 lb. of cement.		Maximum pressure in tons per square foot.
	Fine aggregate.	Coarse aggregate.	
I	$1\frac{1}{2}$	$2\frac{1}{2}$	40
II	$1\frac{7}{8}$	$3\frac{3}{4}$	35
III	$2\frac{1}{2}$	5	30
IV	$7\frac{1}{2}$		20
V	10		15
VI	$12\frac{1}{2}$		10
VII	15		5

ratio be between 6 and 12 the total compressive stress in such wall or pier or such [illegible] exceed a pressure correspondingly proportionate to the pressure appropriate to the slenderness ratios of 6 and 12.

No load-bearing wall or pier constructed of bricks or blocks or of plain concrete nor any storey-height thereof shall have a slenderness ratio exceeding 12 except a cavity wall or a partition wall constructed [illegible] conditions and in accordance with byelaw 45 or 53 respectively.

Provided that where a wall or pier constructed of bricks or blocks or of plain concrete supports a beam or column or is otherwise subjected to local loading of a like nature, and the stresses resulting from such loading are immediately distributed through adjacent material not so stressed, the compressive stress in the material so subjected to local loading may exceed the appropriate maximum pressure indicated in Table VII or Table VIII as the case may be, by not more than 20 per cent.

61.—No account shall be taken of resistance to shearing or tensile stresses in any wall or pier of bricks or blocks or of plain concrete, and such materials shall not be relied upon to resist such stresses except in the case of arches, lintels, corbelling, footings and the like constructions wherein the resistance of the bricks, blocks or plain concrete to shearing and tensile stresses may be deemed to be one-tenth of that to compression.

Provided that if a wall constructed of bricks or blocks or of plain concrete be laterally supported by buttressing walls, piers or other constructions to the satisfaction of the district surveyor and the length of wall between such supports does not exceed 45 feet and does not exceed forty-five times the thickness of such wall, then such wall may be deemed to transmit to such supports a horizontal load equal to 25 per cent. of the wind-pressure on such wall, and for the purpose of calculating the over-turning moment on such wall, the horizontal force due to wind-pressure shall be deemed to be reduced thereby to 75 per cent. of that specified in byelaw 6.

62.—Underpinning shall be carried out to the satisfaction of the district surveyor and in such manner that the permissible stresses prescribed by these byelaws will not be exceeded.

RETAINING WALLS

Definition.—Walls of masonry, concrete, ferro-concrete or brickwork built to retain masses of water or earth are known as retaining walls, and are classified as follows:—Those made (1) to support the pressure of water are termed dams; (2) to keep in safe equilibrium masses of earth, and to prevent sliding action, are known as revetment walls. A breast wall is a particular kind of revetment wall, constructed simply to protect from the weather a freshly exposed surface of a cutting, when the latter is such as to be capable of standing by itself, *i.e.*, when the face has an inclination equal or nearly equal to the natural slope.

Conditions of Stability.—All retaining walls must satisfy the three conditions given previously for the stability of walls in uncemented blocks.

DAMS

Pressure of Fluids.—The laws of fluid pressure state that the pressure at any point varies directly as the depth of that point, and is perpendicular to any surface. Thus,

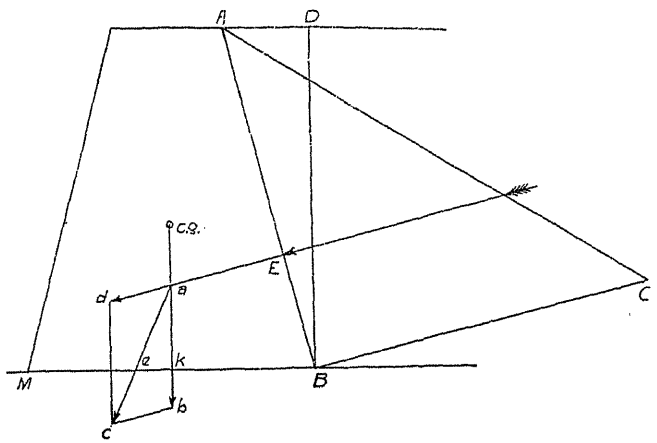


Fig. 181.

against any surface, as AB in Fig. 181, the pressure varies from zero at A to a pressure at B equal to the height BD multiplied by the density of water. Thus if the weight of a cubic foot of fresh water be taken as 62·4 lbs., then

$$\text{pressure at B} = 62{\cdot}4 \text{ BD lbs./sq. ft.}$$

Set out a line BC at right angles to AB to represent this pressure. Join AC. Then the total pressure on unit length of dam is represented by the area of the triangle ACB.

$$\text{Total pressure} = \frac{62{\cdot}4 \text{ BD. AB}}{2} \text{ lbs.}$$

This pressure acts at a point E distance $\frac{1}{3}$. AB from B.

Produce the line of action of the pressure through E and draw the parallelogram of forces. In the diagram, *ab* represents the weight of the wall and *ad* the pressure of the water. The point where the resultant *ac* cuts the base line MB is where the centre of pressure cuts the base of the dam.

Revetment or Retaining Walls.—In the design of these walls it is necessary to determine (1) the pressure due to the earth ; (2) the mass of the wall required to resist the thrust of the earth. The thrust of the earth is an indeterminable quantity. Many theories have been formulated, but they are all approximations. There are two theories which are commonly used : (1) the wedge theory ; (2) Rankine's theory.

The following symbols are used throughout with reference to retaining walls.

ϕ = angle of repose of the earth.
t = thickness of base or bed joint of wall.
h = height of wall in feet.
w_1 = weight of retained earth per cubic foot.
w_2 = weight of wall material per cubic foot.
σ = angle of surcharged earth.
s = ratio of height to the distance set back in the face batter of a wall.
α = angle between f_x and the resultant.

Wedge Theory.—The wedge theory assumes a compact wedge-shaped mass of earth to break away and slide down and forwards. With reference to Fig. 182 let AB be the vertical back of the retaining wall.

Let BC bisect the angle ABD. BC is called the plane of rupture. The triangular prism ABC is the sliding wedge and the angle $ABC = \frac{90 - \phi}{2}$. The pressure of the wedge, represented by GE, acts through the centre of gravity of ABC and intersects CB in E. If BC were perfectly smooth the reaction would be along the normal EH. However, due to friction the reaction acts along EF at an angle ϕ to EH. If GE be made proportional to the weight of the wedge, FG is proportional to the horizontal component of the

weight, which, acting at E, tends to overturn the wall. This force acts at a distance $\frac{1}{3} . h$ from B.

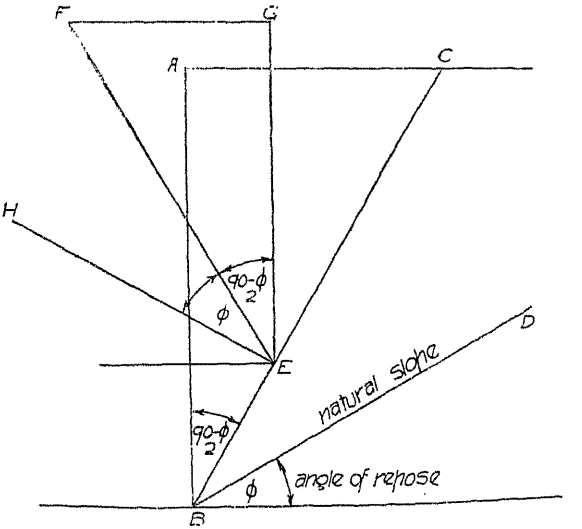


Fig. 182.

$$\text{Overturning moment due to earth} = \text{FG} \times \frac{h}{3}$$

Now $$\text{FG} = \text{GE} \tan \frac{90 - \phi}{2}$$

and since GE is proportional to weight of prism

$$\text{FG} = \frac{w_1 h^2}{2} . \tan \frac{90 - \phi}{2} . \tan \frac{90 - \phi}{2}$$

$$\therefore \text{ Overturning moment} = \frac{w_1 h^3 \tan^2 \frac{90 - \phi}{2}}{6} \quad \ldots\ldots \quad (1)$$

Rankine's Theory.—Rankine's theory is based upon the theory of principal stresses, in which all the stresses acting upon a material are reduced to two principal stresses acting

at right angles one to the other. One of these stresses is the greatest stress to which the material is subjected.

If a mass of earth be banked up with a vertical face, it will slip until it reaches the angle of equilibrium or the angle of repose. This angle is the maximum angle of obliquity which any resultant force can make to the normal to the plane across which the resultant is acting.

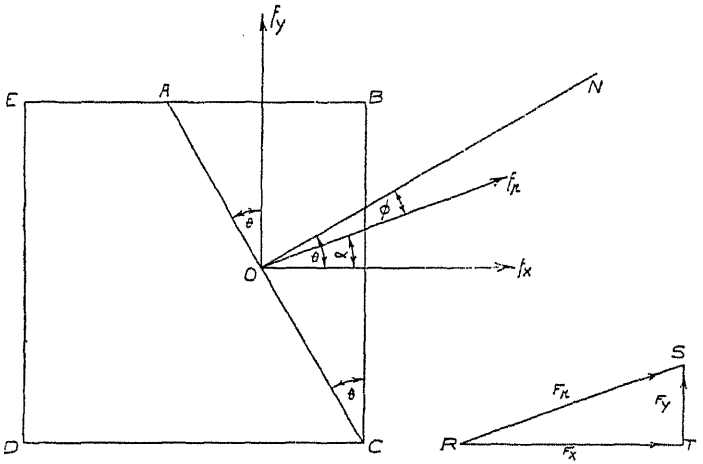


Fig. 183.

The principal stresses are usually denoted by f_x and f_y, and the resultant stress to which they give rise by f_r.

Consider a small cube of material ABCDE acted upon by two principal stresses f_x and f_y (Fig. 183).

Let AC be any internal face and f_r the resultant stress which acts across it. Then if F_x, F_y and F_r be the total forces acting on the faces BC, AB and AC, then $F_x = f_x . BC$, $F_y = f_y . AB$, $F_r = f_r . AC$.

Draw the triangle of forces RST. Then

$$F_r^2 = F_x^2 + F_y^2$$

$$f_r^2 AC^2 = f_x^2 BC^2 + f_y^2 AB^2$$

$$\therefore f_r^2 = f_x^2 \cos^2\theta + f_y^2 \sin^2\theta \quad . \quad . \quad . \quad . \quad . \quad . \quad (2)$$

The vertical component of f_r is $f_y \sin\theta$, and the horizontal component is $f_x \cos\theta$. Hence, f_r makes an angle with the direction of f_x such that

$$\tan\alpha = \frac{f_y \sin\theta}{f_x \cos\theta} = \frac{f_y}{f_x}\tan\theta = m\tan\theta \quad \ldots\ldots\ldots \quad (3)$$

The angle ϕ is the angle between the resultant and the normal ON. It is required to find a relation between f_x, f_y and ϕ when ϕ has its maximum value.

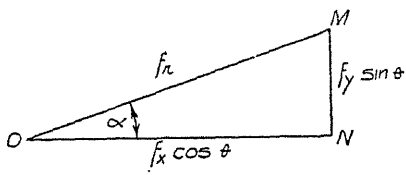


Fig. 184.

From the figure

$$\alpha + \phi + 90 - \theta = 90$$
$$\therefore\ \phi = \theta - \alpha$$

Hence $\tan\phi = \tan(\theta - \alpha) = \dfrac{\tan\theta - \tan\alpha}{1 + \tan\theta\tan\alpha}$

From equation (3) $\tan\alpha = m\tan\theta$.
Then substituting for $\tan\alpha$ we get

$$\tan\phi = \frac{\tan\theta\,(1-m)}{1 + m\tan^2\theta} \quad \ldots\ldots\ldots \quad (4)$$

The angle ϕ is a maximum when $\tan\phi$ is a maximum. To obtain the maximum value of $\tan\phi$ differentiate equation (4) with respect to θ and equate to zero.

$$\frac{d\tan\phi}{d\theta} = \frac{(1 + m\tan^2\theta)(1-m)\sec^2\theta - 2m\tan\theta\sec^2\theta(1-m)\tan\theta}{(1 + m\tan^2\theta)^2}$$

$$0 = \frac{(1-m)\sec^2\theta\left\{1 + m\tan^2\theta - 2m\tan^2\theta\right\}}{(1 + m\tan^2\theta)^2}$$

$$0 = \frac{(1-m)\sec^2\theta\,(1 - m\tan^2\theta)}{(1 + m\tan^2\theta)^2}$$

In order that the above equation should be satisfied we have

$$(1 - m \tan^2 \theta) = 0$$

$$\therefore \tan \theta = \frac{1}{\sqrt{m}} \quad \ldots\ldots\ldots (5)$$

The above is the value of θ when the angle ϕ is a maximum. Substitute this value in equation (4).

$$\tan \phi = \frac{\tan \theta (1 - m)}{1 + m \tan^2 \theta} = \frac{\frac{1}{\sqrt{m}}(1 - m)}{2}$$

$$\tan \phi = \frac{1 - m}{2\sqrt{m}}$$

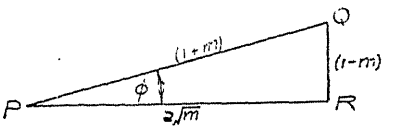


Fig. 185.

From the figure PQR, and knowing the properties of a right-angled triangle, the value of sin ϕ is deduced.

$$\therefore \quad \sin \phi = \frac{1 - m}{1 + m} \text{ and since } m = \frac{f_x}{f_y}$$

$$\sin \phi = \frac{1 - \frac{f_x}{f_y}}{1 + \frac{f_x}{f_y}}$$

$$\sin \phi = \frac{f_x - f_y}{f_x + f_y} \quad \ldots\ldots\ldots (6)$$

By cross multiplying we get

$$\frac{f_x}{f_y} = \frac{1 + \sin \phi}{1 - \sin \phi} \quad \ldots\ldots\ldots (7)$$

Consider a small cube of earth supported by a retaining wall as in Fig. 186. The stresses on it will be (1) a vertical pressure due to the weight of the earth, and (2) a horizontal thrust. These stresses may be considered as principal stresses, and since f_x is greater than f_y, make f_x the vertical stress. The maximum obliquity or the greatest angle that

can occur between the resultant of these two stresses and the normal to the plane across which the resultant is acting is equal to the angle of repose of the earth. If the angle of repose be substituted for ϕ in formula (7), we obtain a relation between f_x and f_y. Now $f^x = w_1 h$. Hence f_y, or the horizontal thrust can be calculated.

$$\therefore \quad \text{The horizontal thrust } f_y = f_x \cdot \frac{1 - \sin\phi}{1 + \sin\phi}$$

$$= w_1 h \cdot \frac{1 - \sin\phi}{1 + \sin\phi}$$

The horizontal thrust is proportional to h, or the depth, and hence the distribution of thrust over the back of the

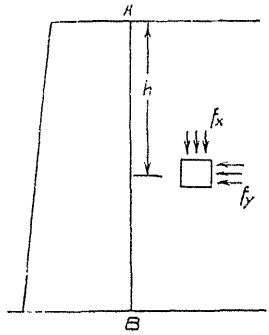


Fig. 186.

wall is a triangle. The total pressure F_y acts at a distance $\frac{1}{3} \cdot h$ from B, and its magnitude is represented by the area of the triangle ABC, Fig. 187,

$$\text{or Total earth pressure } F_y = \frac{w_1 h^2}{2} \cdot \frac{1 - \sin\phi}{1 + \sin\phi} \quad . \quad . \quad . \quad (8)$$

The following table gives the value of the factors in the above formulæ for angles of repose varying from 10 to 45 degrees. The table on p. 328 gives the figures for the angle of repose and the coefficient of friction for the common earths with that for brick and stone.

Angle of repose ϕ	Coefficient of friction Tan ϕ	Sin ϕ	$\left(\frac{1-\sin\phi}{1+\sin\phi}\right)$	$\left(\frac{1+\sin\phi}{1-\sin\phi}\right)$	$\left(\frac{1-\sin\phi}{1+\sin\phi}\right)^2$	$\left(\frac{1+\sin\phi}{1-\sin\phi}\right)^2$
10°	0·176	0·173	0·700	1·425	0·490	2·030
15°	0·267	0·258	0·58	1·725	0·442	2·87
20°	0·363	0·342	0·49	2·04	0·24	4·16
25°	0·466	0·422	0·406	2·46	0·165	6·06
30°	0·577	0·500	0·333	3·00	0·111	9·00
35°	0·700	0·573	0·271	3·69	0·073	13·36
40°	0·839	0·642	0·218	4·58	0·043	21·00
45°	1·000	0·707	0·172	5·82	0·029	34·80

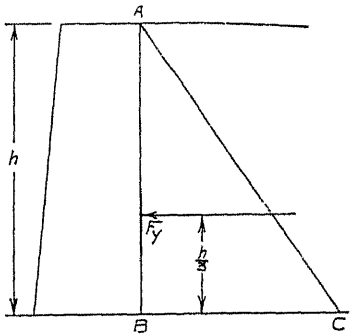


Fig. 187.

Surcharged Retaining Wall.—In employing the Rankine theory in the case of a surcharged retaining wall a different thickness is obtained to that obtained by the wedge method. A particular case is calculated in detail in Example 3, and from this example it appears that it is more economical to employ the Rankine theory.

Rankine Theory for Surcharged Retaining Wall.—Professor Rankine states that in dealing with the case of a surcharged retaining wall—

1. The pressure on any plane parallel to the ground slope is vertical and of intensity equal to $w_1 h$.

2. The stress on any vertical plane is parallel to the surface.

This state of stress is indicated on the small section of material shown in Fig. 188. Thus we have a pair of conjugate stresses f_1 and f_2. The total resultant thrust F is parallel to the surface, and acts at a distance $\frac{1}{3} . h$ from B.

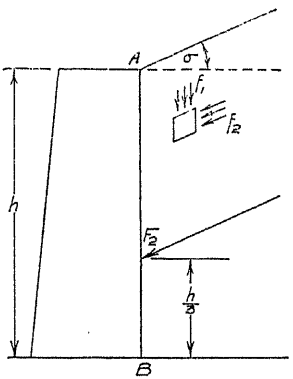


Fig. 188.

Professor Rankine gives the following formulæ for determining F and its maximum intensity f:—

$$= w_1 h \cos \sigma . \frac{\cos \sigma - \sqrt{\cos^2\sigma - \cos^2\phi}}{\cos \sigma + \sqrt{\cos^2\sigma - \cos^2\phi}}$$

$$F = \frac{w_1 h^2}{2} \cos \sigma . \frac{\cos \sigma . - \sqrt{\cos^2\sigma - \cos^2\phi}}{\cos \sigma . + \sqrt{\cos^2\sigma - \cos^2\phi}} \quad . \quad (9)$$

Types of Retaining Walls

Moment of Resistance of Wall.—This is the product of its weight and its leverage. The leverage is measured from the limiting position determined upon for the centre of pressure to a vertical line through the centre of gravity of the wall and transversely along the joint at which failure would be most likely to occur. This is usually at the ground line.

The value of the moment of resistance will depend upon

the form and disposition of the wall. There are five simple types as shown in Fig. 189. (1) The vertical rectangular section. This, though an effective is not an economical section. As the pressure increases with the height of the wall, it is advantageous to increase the width of the base. (2) The vertical back wall with sloping back. In this the increased area of the base gives greater stability and a greater area over which to distribute the pressure. An additional advantage is that the weight of the earth on the

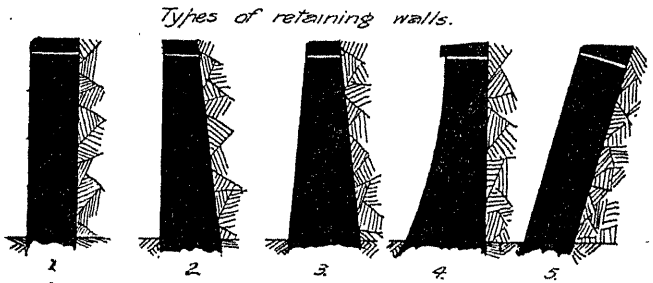


Fig. 189.

wall side of the vertical line AB acts with the weight of the wall. (3) The section with the sloping back and battered face. This offers no practical advantages over the previous case but the battered face gives an appearance of greater stability. (4) The curved battered face. There is no advantage in a curved batter over a straight one, but there is considerable extra expense in building. (5) The sloping rectangular section. This section is wasteful, as that portion of the wall on the earth side of the vertical line drawn from the inside corner of the base does not act with the wall.

The general statement for the moment of resistance of retaining walls is

$$M_R = \text{weight} \times \text{volume} \times \text{leverage}.$$

In all calculations concerning retaining walls, it is usual to consider unit length of the wall. Then we get—

$$\text{Volume} = \text{area of cross-section} \times \text{unity}.$$

In the case of vertical backed walls the thickness may be determined mathematically from the data available, or in other cases, where the weight of the retained earth and the wall material are equal. In other cases it is easier to assume a thickness for the wall, and test for stability either mathematically or graphically.

The statement for the moment of resistance of the wall in No. 1 case is

$$M_R = w_2 ht \times \text{leverage.}$$

The leverage will vary according to the position determined upon for the centre of pressure on the base. If it is desired that the whole base shall be just under compression it would equal $\frac{t}{6}$.

The statement then becomes

$$M_R = w_2 ht \times \frac{t}{6} = \frac{1}{6} \cdot w_2 ht^2$$

The statement for walls with a batter on face and straight back becomes

$$M_R = w_2 h \left\{ \frac{t + t - \frac{h}{s}}{2} \right\} \times \frac{t}{6}$$

$$= \frac{w_2 ht}{12s} \left\{ st2 - h \right\}$$

EXAMPLE I. (Fig. 190).—Let it be required to support a bank of earth, 20 feet in height, weighing 120 lbs. per cubic foot, the angle of repose 30 degrees, by a brick wall with a vertical face and a sloping stepped back weighing 120 lbs. per cubic foot. The centre of pressure is to fall one-sixth of the thickness from the outer edge of the wall, that is, the leverage of the wall will be $\frac{t}{3}$. Factor of safety against slipping not to be less than 1·25.

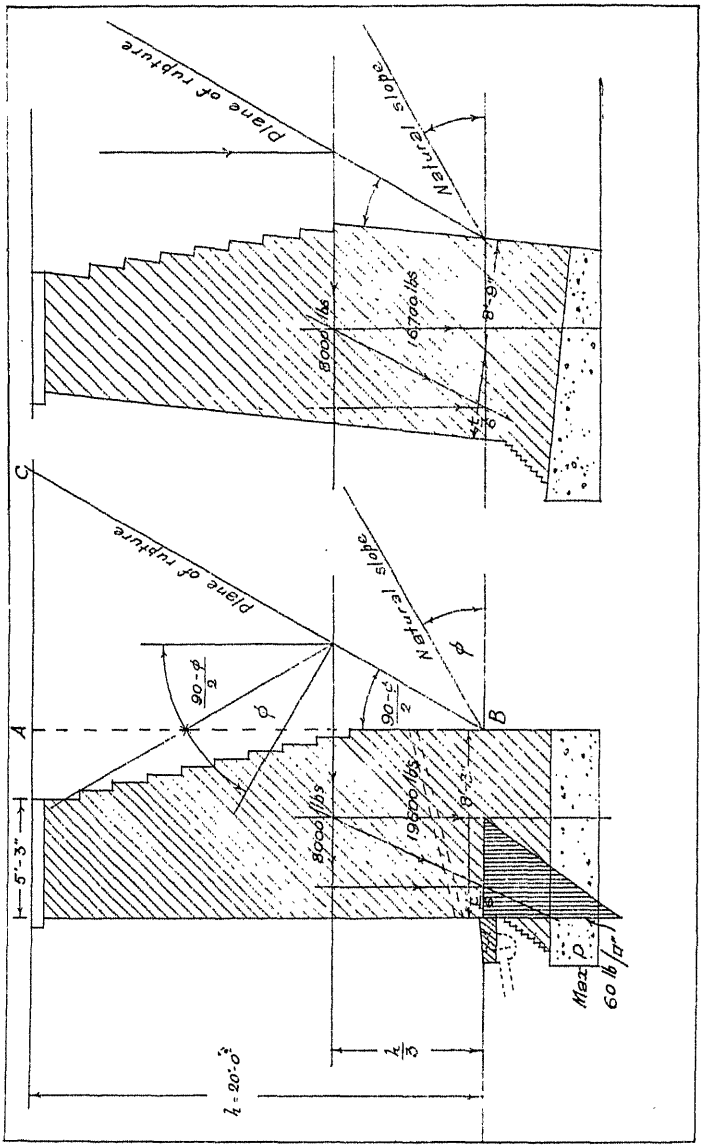


Fig. 190. Fig. 191. Figs. 190—191.

Employ the wedge theory.

$$\text{The overturning moment due to earth} = \frac{w_1 h^3 \tan^2 \frac{90 - \phi}{2}}{6} \cdot \quad (1)$$

$$= \frac{120 \times 20^3 \times \tan^2 \frac{1}{2}(30^\circ)}{6}$$

$$\text{Overturning moment} = 53{,}300 \text{ lbs. ft.}$$

$$\text{Moment of resistance of wall} = \frac{w_2 h t^2}{3}$$

$$= \frac{120 \times 20 \times t^2}{3}$$

$$= 800\, t^2 \text{ lbs. ft.}$$

Now

Moment of resistance = overturning moment.

$$\therefore \quad 800\, t^2 = 53{,}300$$

$$\therefore \quad t = 8{\cdot}15 \text{ ft.}$$

The most suitable even brick dimension is 8 ft. 3 in. as shown in Fig. 190. Since the leverage is $\frac{t}{3}$ only half of the base is under compression. Hence the maximum pressure on the outer edge is twice the average pressure.

$$\text{Average pressure} = \frac{\text{Weight of wall}}{\frac{1}{2} \text{ area of base}}$$

$$= \frac{120 \times 20 \times 8{\cdot}15}{4{\cdot}075 \times 144} \text{ lbs./sq. in.}$$

$$= 30 \text{ lbs./sq. in.}$$

$$\therefore \text{ Maximum pressure} = 2 \times 30$$

$$= 60 \text{ lbs./sq. in.}$$

Coeff. of friction against slipping = 0·7 (see Table, p. 302).

∴ Factor of safety against slipping is

$$\frac{{\cdot}577}{\tan cab} = 1{\cdot}4$$

Employ Rankine's theory.

The overturning moment due to the earth

$$= \frac{wh^3}{6}\left(\frac{1 - \sin \phi}{1 + \sin \phi}\right)$$

$$= \frac{120 \times 20^3}{6} \times {\cdot}333 = 53{,}300 \text{ lbs. ft.}$$

It should be noted that this value for the overturning movement is exactly the same as that calculated by the wedge theory.

EXAMPLE II. (Fig. 191).—Let it be required to support a bank of earth 20 feet high, weighing 120 lbs. per cubic foot, the angle of repose 30 degrees by a brick wall with a batter on the face of 1 in 10 and back approximately vertical. The weight of the brickwork is 120 lbs. per cubic foot. Let the wall act with a leverage of $\frac{t}{3}$. The factor of safety against slipping not to be less than 1.25.

$$\text{Overturning moment} = \frac{w_1 h^3}{6} \cdot \left(\frac{1 - \sin\phi}{1 + \sin\phi}\right)$$

$$= \frac{120 \times 20^3}{6} \times \cdot 333$$

$$= 53{,}300 \text{ lbs. ft.}$$

$$\text{Moment of resistance of wall} = w_2 h \left(\frac{t + t - \frac{h}{10}}{2}\right)\frac{t}{3}$$

$$= \frac{w_2 h t}{6}(2t - 2)$$

$$= \frac{120 \times 20 \times t}{6}(2t - 2)$$

$$= 800\,t^2 - 800\,t$$

Equate the two moments

$$800\,t^2 - 800\,t - 53300 = 0$$

$$t = \frac{800 \pm \sqrt{800^2 + 4 \times 53300 \times 800}}{2 \times 800}$$

$$= 8{\cdot}65.$$

The nearest even dimension is 8 ft. 9 in. as shown in Fig. 191. As the earth in this case is of the same weight as the brickwork, the back of the wall might be economically sloped, as shown in the figure.

Produce the line of thrust of the earth and draw the parallelogram of forces. *ab* represents the weight of the wall, 12,300 lbs., and acts through its centre of gravity, and *ad* the thrust of the earth, and equals 4,125 lbs. *gc* is the resolved component of the resultant *ac*, and acts at right angles to the bed joint on the point *e*.

Coeff. of friction against sliding = 0·7 (see Table, p. 302).

$\therefore$ Factor of safety $= \frac{0{\cdot}7}{\tan gca} = 1{\cdot}2$ approx.

This would be safe, as it would be assisted by the earth in front of the concrete and footings.

To determine the maximum pressure on the bed joint.

As the resultant pressure intersects the bed joint at $\frac{t}{3}$ from the centre of the base, the whole of the weight of the wall will be taken on one-half of the base. That is on an area of 4·325.

Then the average pressure $= \frac{W}{A} = \frac{16700}{4{\cdot}325 \times 144}$
$= 26{\cdot}5$ lbs. per square inch.
Maximum pressure $= 2$ average pressure.
$= 2 \times 26{\cdot}5$
$= 53$ lbs. per square inch.
(See Fig. 191.)

EXAMPLE III. (Fig. 192).—Let it be required to retain a bank of earth 20 feet in height, angle of surface slope 25 degrees, angle of repose 30 degrees, weighing 120 lbs. per cubic foot, by a retaining wall with a vertical face and a sloping stepped back in brickwork. The brickwork is to weigh 120 lbs. per cubic foot. Let the earth encroach beyond the theoretical back AB a distance of $\frac{h}{6}$.

Assume

1. Leverage of wall not greater than $\frac{t}{3}$
2. Maximum pressure on bed joint not to exceed 80 lbs./sq. in.
3. Factor of safety against slipping not less than 1·25.
4. Thickness of wall at base to be 8 ft. 3 in.

Employ the Rankine theory. In using this theory for the case of a surcharged wall it is necessary to assume a thickness to the wall, and then test it for the three methods of failure (1) overturning, (2) crushing, (3) sliding. This assumption is necessary because of the impossibility otherwise of determining the position of the centre of gravity of the wall except in the case where the cross section is to be vertical and rectangular.

Consider earth pressure on AB. From Fig. 192.

$$AB = h + \frac{h}{6}\tan\sigma = 20 + \frac{20}{6}\tan 25 = 21{\cdot}55 \text{ ft.}$$

$$F = \frac{w_1 AB^2}{2} \cdot \cos\sigma \cdot \frac{\cos\sigma - \sqrt{\cos^2\sigma - \cos^2\phi}}{\cos\sigma + \sqrt{\cos^2\sigma - \cos^2\phi}} \quad \text{. . . . . see (9)}$$

$$= \frac{120 \times 21{\cdot}55 \times 21{\cdot}55 \times 0{\cdot}9063}{2} \cdot \frac{0{\cdot}9063 - \sqrt{0{\cdot}9063^2 - 0{\cdot}866^2}}{0{\cdot}9063 + \sqrt{0{\cdot}9063^2 - 0{\cdot}866^2}}$$

$$= 13{,}850 \text{ lbs.}$$

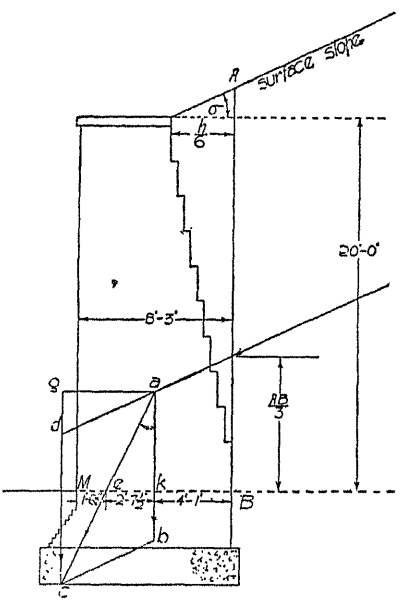


Fig. 192.

This force acts at a distance $\frac{AB}{3} = 7{\cdot}18$ feet from B and in a direction parallel to the surface slope. Now proceed graphically.

From Fig. 192 we get that the weight of the wall acts through a vertical line distance 4 ft. 1 in. from AB.

$$\text{Weight of wall} = w^1 \left\{ ht + \frac{1}{2} \cdot \frac{h}{6} \cdot \frac{h}{6} \tan 25 \right\}$$

$$= 120 \left\{ 20 \times 8{\cdot}25 + \frac{400 \times 0{\cdot}4663}{72} \right\}$$

$$= 20{,}100 \text{ lbs.}$$

Produce the line of thrust due to the earth. At the point *a*, where it cuts the line through the centre of gravity, draw *ab* = 20,100 lbs. Make *ad* = 13,850 lbs. Then *ac* represents the resultant, and *gc* is the vertical component of the resultant. This vertical component acts on the point *e*.

Consider conditions required. From Fig. 192:

1. Leverage equals $ek = 2'\ 7\frac{1}{2}''$, which is less than $\frac{t}{3}$

2. Coeff. of friction against sliding = 0·7 (see Table, p. 302).

$$\therefore \text{ Factor of safety} = \frac{0{\cdot}7}{\tan cab} = \frac{0{\cdot}7}{0{\cdot}4838}$$

$$= 1{\cdot}45$$

3. Maximum pressure. The pressure *gc*, which acts at *e*, is spread over a length of base equal to 3M*e*.

$$\text{Now } Me = 1'\ 6\tfrac{1}{2}'' = 1{\cdot}54 \text{ ft.}$$

$$\therefore \quad 3Me = 4{\cdot}62 \text{ ft.}$$

$$\text{Average pressure} = \frac{ge}{4{\cdot}62 \times 144}$$

$$= \frac{25800}{4{\cdot}62 \times 144}$$

$$= 38{\cdot}8 \text{ lbs./sq. in.}$$

$$\therefore \text{ Maximum pressure at M} = 2 \times 38{\cdot}8$$

$$= 77{\cdot}6 \text{ lbs./sq. in.}$$

EXAMPLE IV. (Fig. 193).—Let it be required to sustain a bank of earth 20 feet in height, angle of surface slope 25 degrees, angle of repose 30 degrees, weighing 120 lbs. per cubic foot, by a concrete retaining wall with a batter on face of 1 in 10, back approximately vertical, and weighing 150 lbs. per cubic foot.

Assume

1. Leverage of wall not greater than $\frac{5}{12}\ t$

2. Maximum pressure on bed joint not to exceed 80 lbs./square inch.

Employ the formula given by Rankine. Consider earth pressure on AB.

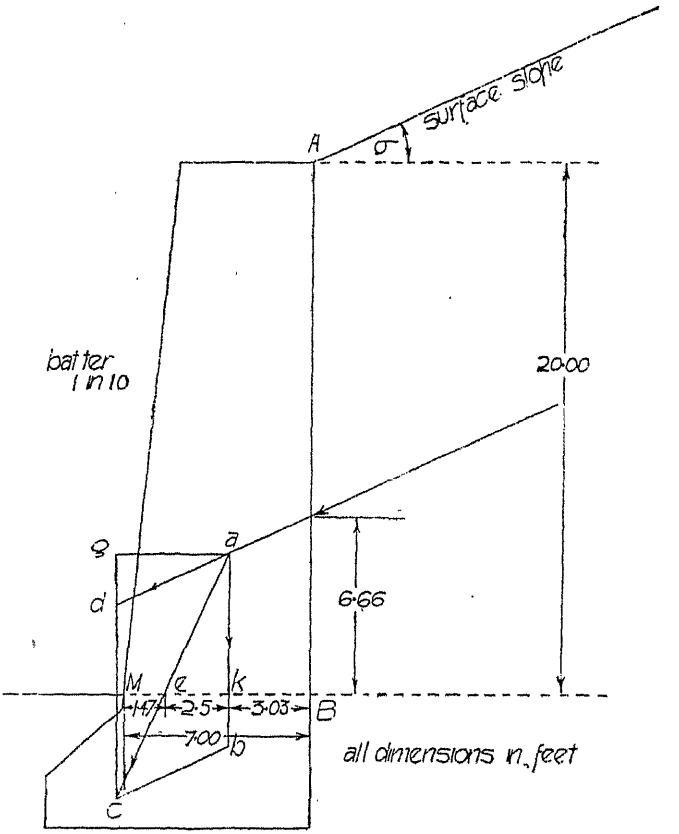


Fig. 193.

$$F = \frac{w_1 h^2}{2} \cdot \cos\sigma \frac{\cos\sigma - \sqrt{\cos^2\sigma - \cos^2\phi}}{\cos\sigma + \sqrt{\cos^2\sigma - \cos^2\phi}}$$

$$= \frac{120 \times 20 \times 20 \times 0{\cdot}9063}{2} \cdot \frac{0{\cdot}9063 - \sqrt{0{\cdot}9063^2 - 0{\cdot}866^2}}{0{\cdot}9063 + \sqrt{0{\cdot}9063^2 - 0{\cdot}866^2}}$$

$$= 11{,}940 \text{ lbs.}$$

Assume a thickness of 7 feet for the base of the wall.

$$\text{Weight of wall} = w_2 h \left\{ \frac{t + t - \frac{h}{10}}{2} \right\}$$

$$= \frac{150 \times 20}{2} \times 12 = 18{,}000 \text{ lbs.}$$

This weight acts through the centre of gravity at a distance 3·03 feet from AB.

Draw the parallelogram of forces and consider the conditions required to prevent failure.

1. Leverage = 2·5 ft. which is less than $\frac{5}{12} t$

2. Maximum pressure. The resolved vertical component of the resultant, *i.e.*, *gc*, cuts the bed joint at *e*. From Fig. 193, M*e* = 1·47 ft. Now

Length of base over which pressure is spread = 3M*e*
= 4·41 ft.

$$\therefore \text{ Average pressure} = \frac{gc}{4{\cdot}41 \times 144}$$

$$= \frac{22800}{4{\cdot}41 \times 144}$$

$$= 36 \text{ lbs./square inch}$$

$$\therefore \text{ Maximum pressure at M} = 2 \times 36$$

$$= 72 \text{ lbs./square inch.}$$

If the wedge method be employed to find a thickness for this wall the answer works out at 10 feet. This is 3 feet more than that found to be necessary by the above method.

Generally.—Cuttings through clay swell if exposed to the air, and exert a force on the back of the wall which is difficult to determine. The thickness of the latter must be ascertained by experience.

In calculating for retaining walls, the angle of repose of the earth to be supported must be either known or determined by experiment (the earth being in that state in which by proper drainage it will remain), likewise the weights of the different materials. The former is given on p. 302, the latter on p. 214.

Should the retained earth be such that when saturated with water it is practically mud, the retaining wall must be

calculated as a dam to resist fluid pressure. The materials usually employed are concrete, brick, and masonry, thoroughly good bond being important in both of the latter.

The backs should be left rough or built in steps, and a layer of loose stone at least 12 inches in thickness, gravel or other porous material to afford a passage of water to the weep holes, should, in retentive soils, be packed up behind the walls, and weep holes not less than 7 square inches in sectional area placed along the bottom, from 5 to 10 feet apart; and in the case of retentive soils, one weep hole to every 4 square yards of face. The weep holes must be connected to a surface drain to protect the foundations and footings, as shown in Figs. 190 and 191. In the section of wall usually adopted in practice, the thickness for about one-third the height from the base, is made equal to $\frac{h}{3}$ to $\frac{h}{4}$, and is reduced towards the top in regular offsets at the back.

The face is generally made to batter from 1 in 6 to 1 in 10. Too great a batter in a retaining wall is not desirable, as the wet gets into the joints, and tends to destroy the wall, unless they are pointed with cement.

A retaining wall with a batter of 1 in 10 can be bonded easily into an adjacent wall with a vertical face without inconveniently thickening the joints.

A batter is desirable, as it economically adds to the stability of the wall, and in the event of any slight outward displacement, the wall is not rendered apparently unsafe, as would be the case with a vertical face.

Piers.—Attention has been drawn on p. 291 to the advisability of loading piers axially in order to obtain uniformity of stress on the base. As this cannot always be done, it becomes necessary to enquire as to whether the eccentricity of the load will dangerously increase the pressure on any portion of the base.

The loads on a pier will consist of the applied loads plus the weight of the pier. Let the applied load be W_1, and the

weight of the pier be W_2. Then the statement for the intensity of pressure per square inch f on the base will be—

$$f = \frac{W_1}{A}\left(1 \pm \frac{ey}{r^2}\right) \pm \left(\frac{W_2}{A}\right).$$

Example.—A pier 10 feet high, 3 feet wide, 2 ft. 3 in. thick, supports one end of a girder which transmits a load of 30 tons to the pier, length of bearing 1 ft. 2 in. Weight of brickwork 120 lbs. per cubic foot; $r^2 = 60{\cdot}75$ inches, eccentricity 6·5 inches.

$$\text{Then } f = \frac{W_1}{A}\left(1 \pm \frac{ey}{r^2}\right) \pm \frac{W_2}{A}.$$

Weight per square inch on base due to pier

$$= \frac{10 \times 120}{144} = 8{\cdot}3 \text{ lbs. per square inch.}$$

$$= \frac{W_1}{A}\left(1 + \frac{ey}{r^2}\right)$$

$$= \frac{30 \times 2240}{36 \times 27}\left(1 \times \frac{6{\cdot}5 \times 13{\cdot}5}{60{\cdot}75}\right)$$

$$= 168{\cdot}6 \text{ lbs. per square inch.}$$

$$ft = \frac{W_1}{A}\left(1 - \frac{ey}{r^2}\right)$$

$$= \frac{30 \times 2240}{36 \times 27}\left(1 - \frac{6{\cdot}5 \times 13{\cdot}5}{60{\cdot}75}\right)$$

$$= -30{\cdot}404 \text{ lbs. square inch.}$$

Stress at one edge $= 168{\cdot}6 + 8{\cdot}3 = 176{\cdot}9$ lbs./square inch.

Stress at other edge $= 8{\cdot}3 - 30{\cdot}404 = -22{\cdot}104$ lbs./square inch

As $\frac{r^2}{e}$ is less than y part of the base will be in tension, and to determine the distance y_2 where there is no stress, that is, where the stress changes or where $f = 0$

$$y_2 = \frac{r^2}{e}. \quad \text{See page 293.}$$

$$= \frac{60{\cdot}75}{6{\cdot}5} = 9{\cdot}34.$$

The diagram (Fig. 194) shows the nature of the stress on the base.

The Standard Specification gives 16 tons per super foot or 249 lbs. per square inch as the safe load for bricks of the 5,000 to 7,500 lbs. class in cement mortar 1—3; as the greatest intensity of pressure is 176·9 lbs. per square inch, the pier would be safe if built with bricks of this standard.

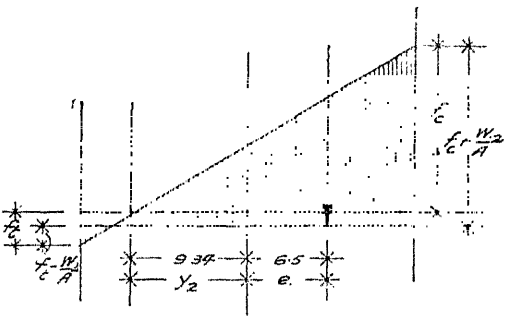


Fig. 194.

Tall Piers.—The height of a pier in brickwork above any horizontal section should not exceed twelve times the least dimension of that section. The area of the base of such piers should be proportioned to the pressure they have to resist. For economy in labour the sides of the piers are usually carried up vertically and have the same sectional area at top as at the bottom, that is, they have an excess of strength and therefore of material. For perfect and economical construction the horizontal sections of a pier at any part should be proportioned to the pressure upon them. It would only be in the case of very tall piers supporting very heavy loads that it would be economical to design the piers to the theoretical sections, but if the piers are sufficiently large to build hollow then the theoretical section may be kept and built to with economy.

EXAMPLE I.—Let it be required to support a load of 50 tons at a height of 30 feet on a brick pier approximately square, the safe load on the brickwork being taken as 6 tons

per square foot, and the weight of brickwork 112 lbs. per cubic foot. Determine the area of the pier.

$$\text{The area of the top course} = \frac{\text{total load}}{\text{safe load}}$$

$$= \frac{50}{6} = 8\tfrac{1}{3} \text{ square feet.}$$

Let A = area of required base
P = area of top base
r = ratio of increase of each block over the block above
R = $(1 + r)$
n = number of unit blocks counting from the top
then $A = P R^n$.

Consider blocks of $1' \times 1' \times 1'$ to weigh 112 lbs.

$$\text{then area of the lower block required to support itself} = \frac{\text{weight of brickwork}}{\text{safe load}}$$

$$= \frac{112}{6 \times 2240} = \frac{1}{120}$$

$$\therefore r = \frac{1}{120} \text{ and } R = \left(1 + \frac{1}{120}\right) = \frac{121}{120}$$

$$A = P R^n$$

$$= \frac{25}{3}\left(\frac{121}{120}\right)^{30}$$

$$= 10{\cdot}69 \text{ square feet.}$$

The top of pier has an area of 8·3 square feet, and the base of lowest course 10·69 square feet, and the area of the base of any intermediate block would be calculated by the same formula.

If it be required to build the sides of this pier vertical an increase in the area of the base would be necessary to support the increased mass to maintain the maximum pressure of 6 tons per square foot. The increased area may be obtained in the following manner :—

Let x = side of required vertical faced pier
x^2 = area of required vertical faced pier
A = area of computed pier
w = weight of brickwork per cubic foot
then $(x^2 - A) \times$ safe load = $x^2 h w$ − weight of computed pier.
S = volume.

The volume of computed pier may be obtained working downwards by the formula for the sum of a geometrical series, viz.—

$$S = P\left(\frac{R^n - 1}{R - 1}\right) = 8{\cdot}3\left(\frac{\left(\frac{121}{120}\right)^{30} - 1}{\frac{121}{120} - 1}\right)$$

$$S = 280{\cdot}125 \text{ cubic feet}$$

Working upwards, by formula

$$S = A\left(\frac{1 - R^n}{1 - R}\right)$$

$$= 10{\cdot}69\left(\frac{1 - \left(\frac{120}{121}\right)^{30}}{1 - \frac{120}{121}}\right)$$

$$= 286{\cdot}95 \text{ cubic feet}$$

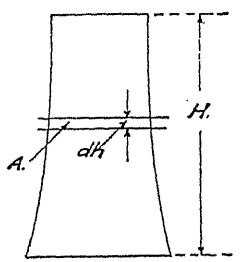


Fig. 195.

Therefore the mean of these two is 283·53 cubic feet, and the weight equals 31,800 lbs.

Then

$$(x^2 - 10{\cdot}69)\ 6 \times 2240 = x^2 \times 30 \times 112 - 31800$$

$$x^2 = \frac{111800}{10080} = 11{\cdot}09$$

If a square pier, then $x = 3{\cdot}33$ feet; the nearest brick dimensions would be $4\frac{1}{2}$ bricks, that is 3 ft. $4\frac{1}{2}$ in.

An expression for the total volume of the pier may be deduced by means of the calculus in the following manner (see Fig. 195).

$$\text{Volume of strip} = A\,dh$$

$$\text{Total volume} = \int_0^H A\,dh$$

$$= \int_0^H PR^h\,dh$$

$$= P\int_0^H R^h\,dh$$

$$= P\left[\frac{R^h}{\log_e R}\right]_0^H$$

$$= \frac{P}{\log_e R}\left\{R^H - 1\right\}$$

$$\text{Total volume} = \frac{P}{2{\cdot}3\log_{10} R}\left\{R^H - 1\right\}$$

In this case

$$\text{Volume} = \frac{8{\cdot}33}{2{\cdot}3\log_{10}\left(\frac{121}{120}\right)}\left\{\left(\frac{121}{120}\right)^{30} - 1\right\}$$

$$= 283{\cdot}67 \text{ cu. ft.}$$

EXAMPLE II.—Let it be required to support a load of 1,000 tons at a height of 100 feet on a brick pier, the safe load on the brickwork being taken as 6 tons per superficial foot, and to be stressed to this amount at any horizontal section, and the weight of the brickwork as 112 lbs. per foot cube.

The dimensions of the pier at top to be 27′ 0″ × 8′ 0″

$$\therefore P = \frac{\text{total load}}{\text{safe load}} = \frac{1000}{6} = 166{\cdot}6 \text{ sup. feet.}$$

at 20′ down $A = P R^{20}$

$$= 166{\cdot}\dot{6} \times \left(\frac{121}{120}\right)^{20} = 196{\cdot}68 \text{ sup. feet.}$$

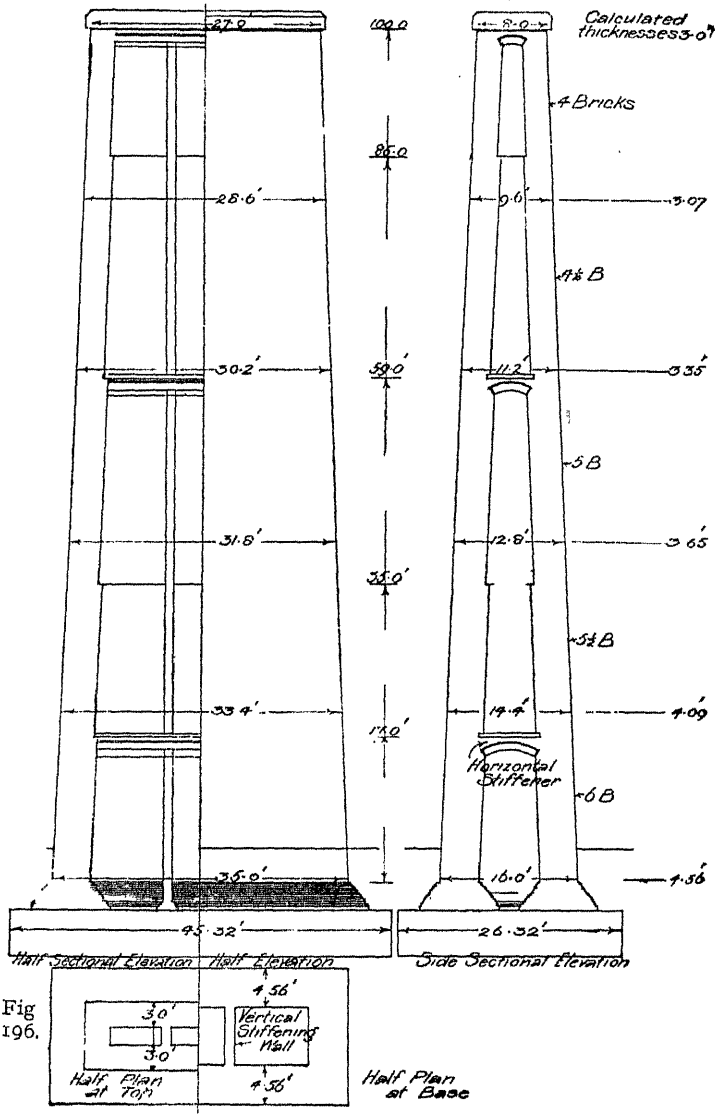
40′ down $A = P R^{40}$

$$= 166{\cdot}\dot{6}\left(\frac{121}{120}\right)^{40} = 232{\cdot}19 \text{ sup. feet.}$$

60′ down $A = P R^{60}$

$$= 166{\cdot}\dot{6}\left(\frac{121}{120}\right)^{60} = 274{\cdot}11 \text{ sup. feet.}$$

Fig. 197. Fig. 198.



Figs. 196—198.

80′ down $A = P R^{80}$

$= 166{\cdot}\dot{6}\left(\frac{121}{120}\right)^{80} = 323{\cdot}6$ sup. feet.

100′ down $A = P R^{100}$

$= 166{\cdot}\dot{6}\left(\frac{121}{120}\right)^{100} = 382{\cdot}03$ sup. feet.

To obtain the varying thicknesses of the pier at each 20 feet in its height. Let the batter be taken as 1 in 25, then commencing at the 100 feet level, the dimensions as shown in Figs. 196 to 198 at—

100	feet	level	= 8·0′ × 27·0′
80	,,	,,	= 9·6′ × 28·6′
60	,,	,,	= 11·2′ × 30·2′
40	,,	,,	= 12·8′ × 31·8′
20	,,	,,	= 14·4′ × 33·4′
0	,,	,,	= 16·0′ × 35·0′

The thickness at the 100 feet level has already been obtained, and is 3 feet, then at the 80 feet level the thickness x may be obtained as follows:—

$$\begin{aligned}
(\text{round of walls} - 4x)\,x &= \text{required area} \\
76{\cdot}4x - 4x^2 &= 196{\cdot}68 \\
x^2 - 19{\cdot}1x &= -49{\cdot}17 \\
x^2 - 19{\cdot}1x + (9{\cdot}55)^2 &= 91{\cdot}212 - 49{\cdot}17 \\
x - 9{\cdot}55 &= 9{\cdot}55 - 6{\cdot}48 \\
x &= 3{\cdot}07
\end{aligned}$$

Then determining the thicknesses at the other levels in a similar manner, the thicknesses required as shown in Fig. 197 will be as follows:—

100	feet	level	...	...	...	... 3·0′
80	,,	,,	...	...	...	... 3·07′
60	,,	,,	...	...	...	... 3·35′
40	,,	,,	...	...	...	... 3·68′
20	,,	,,	...	...	...	... 4·09′
0	,,	,,	...	...	...	... 4·56′

The area of spread for the footings and concrete may be determined as has been previously explained.

The volume of the pier may be determined from the formula given before, namely,

$$\text{Total volume} = \frac{P}{2{\cdot}3 \log_{10} R}\left\{R^H - 1\right\}$$

$$= \frac{166{\cdot}6}{2{\cdot}3 \log_{10}\left(\frac{121}{120}\right)}\left\{\left(\frac{121}{120}\right)^{100} - 1\right\}$$

$$= 25{,}972 \text{ cu. ft.}$$

$$\therefore \text{ Weight} = \frac{112 \times 25972}{2240} \text{ tons.}$$

$$= 1298{\cdot}6 \text{ tons.}$$

Figs. 196, 197 and 198 show the construction of the pier with the necessary stiffening walls and arches.

CHAPTER XIII

FLUES, FIREPLACES AND TALL CHIMNEYS

Chimney Stacks.—Enclosed channels, formed usually in brickwork, to discharge (1) smoke, are termed smoke flues ; (2) vitiated atmosphere, are termed foul air flues ; a construction containing one or more vertical flues being known as a chimney stack.

Theory of Chimney Construction.—For a flue to function satisfactorily, there must be a good draught. The actual velocity of the heated gases in a flue is dependent upon so many factors, that with open fireplaces it is impossible to formulate an expression that would give with any precision either the area of the flue required or the measure of its draught. For boiler work, where the rate of combustion, the kind of fuel employed and the requisite quantity of air to support combustion is under control, approximations may be made as to the height and area of the chimney, but with open grates, experience is the only guide to indicate the points to be observed if a good upward draught is to be obtained.

The upward draught is due to the difference in weight between the column of heated air in the flue and a similar column of cold air. In order that the air in the flue can become readily heated, its area should be kept as small as its requirements necessitate. The higher the shaft or the greater the length of the flue, the greater will be the velocity. The inside of the flue should be uniform in size and shape, and its inside surfaces made as smooth as possible to reduce friction. Theoretically it should be straight, but experience has shown that a bend in the flue is instrumental in stop-

ping down draughts. All bends should be made as easy as possible, and the inclination should never be less than 45 degrees. The interior of all flues should be parged or formed with fireclay linings. One frequent cause of down draughts is the cavernous space so frequently made immediately above the fireplace opening, where the sides are gathered over to the section of the flue. This construction is not necessary with the modern slow-combustion stoves and where the cast-iron or concrete lintel is used in lieu of the rough arch and large void. The external thickness of the flue wall should be at least 9 inches to limit the loss of heat. For residential work the thickness of the internal flue walls is invariably made 4½ inches. Such walls are usually plastered in the rooms, but where they pass through the floors or the roof they should be rendered, to reduce the conduction of the heat, and to seal any faulty joints in the brickwork at those parts. The outlet of flues should be above any adjacent ridge, gable or stack. A small contraction of the flue is said to increase the velocity of the heated gases. This is doubtful, but such a contraction is usually formed by the pots.

Where flues are grouped as in chimney breasts, it is essential that the withs or the divisions between the flues should be properly bonded to the outer walls. In all flues 9 inches and upward, there will be in every alternative course a brick that is not tailed into the outer wall. These should be carefully bedded and the side joints completely filled with mortar to prevent any possibility of any communication between the flues, especially at any bends, as in the process of sweeping these withs are liable to be displaced.

Parging or Pargetting is the process of rendering the inside of flues. This should be done with good cement mortar trowelled smooth. It facilitates the draught by reducing friction between the ascending gases, and is a precaution against any communication between adjoining flues through defective joints in the withs or in the outer walls of the flues. When parging, a small sack filled with shavings or sawdust should be drawn up the flue as the parging proceeds to ensure a clear passage and to prevent

any droppings lodging on the bends and obstructing the passage. This is known as coring. For the best work the flues should be lined with fireclay flue linings (see Fig. 200).

Material for Flues.—The materials used for chimneys must be incombustible, durable, and sufficiently stable to resist external forces, such as wind pressure. Without the assistance of ties or any other special construction, brickwork best satisfies these conditions.

Dimensions of Flues.—Flues 9″ × 9″ are the accepted standard dimensions for flues for ordinary purposes, larger sizes being more liable to down draught and smaller sizes having practical objections connected with cleaning and stoppages. Rectangular 9″ × 9″ and circular 10-inch fireclay flues are used instead of the pargetted lining in brick chimney shafts, and these should always be used in stone shafts. They make a clean lining, and the liability of the withs becoming imperfect is obviated.

Circular, rectangular and square fireclay flue linings are shown in Fig. 200. These are all stock articles and can be obtained from 7 inches diameter upwards. Fig. 202 shows C.I. chimney lintels, to be used with the fireclay linings. These form a very efficient support for the flue linings and do away with the necessity for a relieving arch. They are of cast iron, ½ inch thick, with a strengthening rib and socket for flue.

Classification of Smoke Flues.—The plans of chimneys may be arranged in three ways, viz.: (*a*) Back to back, as shown in Fig. 202; (*b*) side by side, or interlacing, 203; (*c*) diagonally, 204.

Fig. 201 shows the fireplace on an external wall, with 9″ × 9″ flues, with the backs made 4½ inches in thickness—the figure shows the 4½ inches by dotted lines—the Model Bye-Laws stipulate 4 inches. This arrangement is most usually adopted to curtail expense, but it is better work to build the backs 9 inches in thickness.

The interlacing system, as shown in Fig. 203, is sometimes used in order that the projection into the room may be as small as possible; but it could only be adopted in

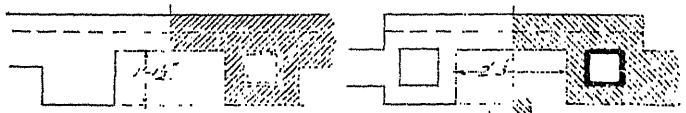


Fig .201 *Fireplaces and Flues.*

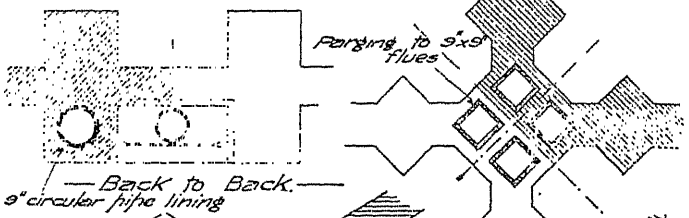


Fig. 202.



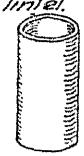






Fig. 199.

Fig. 205.

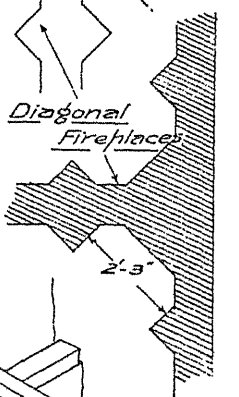


Fig. 204

Interlacing Fireplaces.

Fig. 203.

Figs. 199—205.

party walls, where both houses belong to the same property, as the flues would project beyond the centre line. The diagonal method, as shown in Fig. 204, is more particularly suitable for use in small rooms, where a fireplace near the centre of room would not be convenient. This type may be arranged with one fireplace, or in groups of two, three or four at any one level.

Fig. 205 shows an isometric view of the construction of a fireplace in an upper floor, showing the construction adjacent to the hearth, the position of the arch and the method of gathering over the flues.

Figs. 206 and 207 show the arrangement of a double stack in an external wall, one side arranged with fireclay flue linings and one side the flues pargetted. These views also show the method of gathering over the flues at each level, and attention is directed that where the flues are massed at the upper portion, there is an extra thickness between them. This is to prevent the displacement of bricks when the flues are swept.

Generally.—Every fireplace should have a separate flue, and in a stack the partition walls or withs must be smoke-proof. Any connection between flues causes smoky chimneys.

A grouping of flues in a building tends to economy and effectiveness.

Chimney Bond.—The bonding in chimneys with 4½-inch backs is most effectively accomplished by the use of Flemish bond; this method lends itself to the bricks of the withs of one course forming the face headers, and bats only being used on face in the alternate courses, as shown in Fig. 1117, *Elementary Course.* The use of English bond for 4½-inch backs necessitates a considerable amount of cutting, thus causing the work to be weak and expensive.

Foundations for Chimneys.—The Model Bye-Laws provide that all chimneys must be built upon solid foundations which would comply with the requirements of the bye-law governing the foundations of structural walls if the chimney were a pier forming part of the wall. It must have a damp-proof course if the wall is required to be provided with a

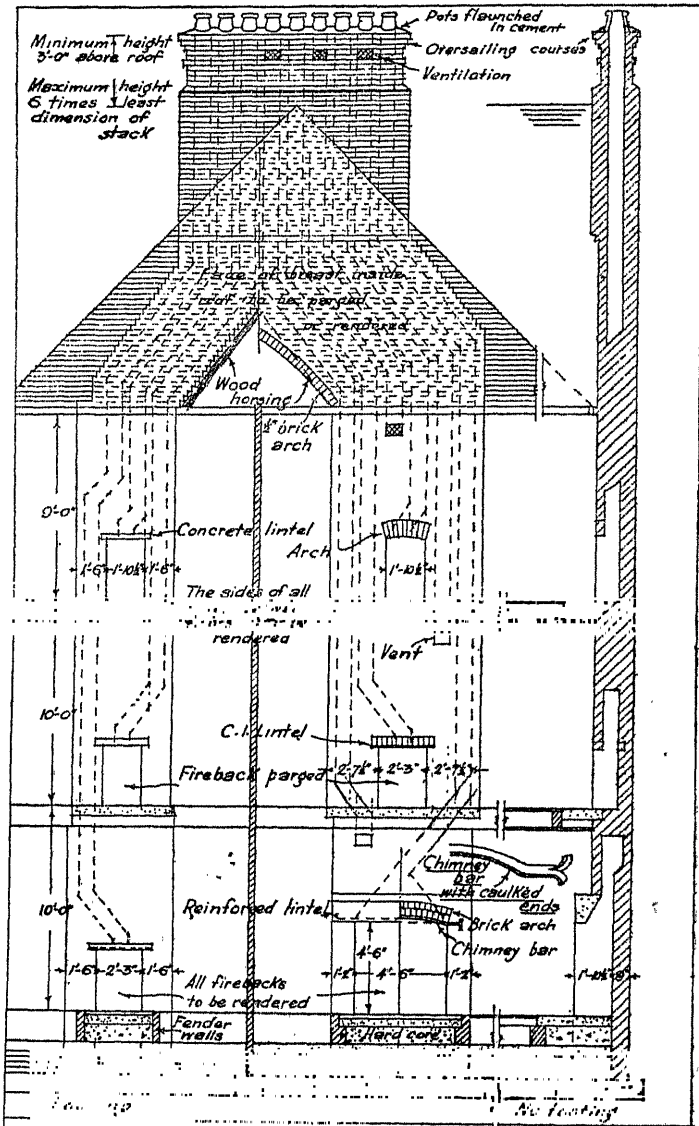


Fig. 206. Fig. 207.

Figs. 206—207.

damp-proof course and must be properly bonded with or otherwise securely tied into the wall. Whether brick footings are used or not will depend on whether the adjacent wall has brick footings or rests direct upon a cement concrete slab of suitable width and thickness (see Fig. 206).

Whereas in the case of a chimney commencing above a shop, it must be built upon sufficient corbels of bricks or stone, or other incombustible substances. This corbelling must not project more than the thickness of the wall below the corbel, otherwise the centre of gravity of the mass will be brought dangerously near the face of the lower part of the wall, and there will be a tendency to overturn.

Fireplace Openings.—Openings for special purposes such as for kitchen ranges must be made of dimensions sufficient for their requirements. The standard dimensions for ordinary open fires are 3′ × 3′ × 1′ 2″. These dimensions became usual when the hob grate was in vogue. The improved slow combustion grates, however, do not require anything like such a large opening. The grate, apart from its surround, consists of a firebrick lump moulded in one or more pieces (see Fig. 213). The best types are of the well pattern. These have a separate base in one or three pieces which is set with its top at the floor level. Its central portion is sunk to form a well. The upper part consists of two sides converging inwards. The back rises vertically from the base for about 6 inches then slopes forward to within about 4 inches from the top, where it slopes back at an angle of about 40 degrees. It is in one piece of firebrick, approximately 2 inches in thickness. The depth of the fire should be at least 10 inches. The blocks are made in various widths from 12 to 21 inches. This, with the thickness of the sides, requires openings from 16 to 25 inches. The openings in the breasts, which should be made some multiple of a half brick, would vary from 18 to 27 inches. This represents a considerable economy in the width required for the breasts and a saving in floor area in the rooms.

The sloping sides of the cheeks of the blocks reflect the heat laterally into the room, and the back leaning forward reflects the heat downward on to the floor. In

Fig. 209

Fig. 208.

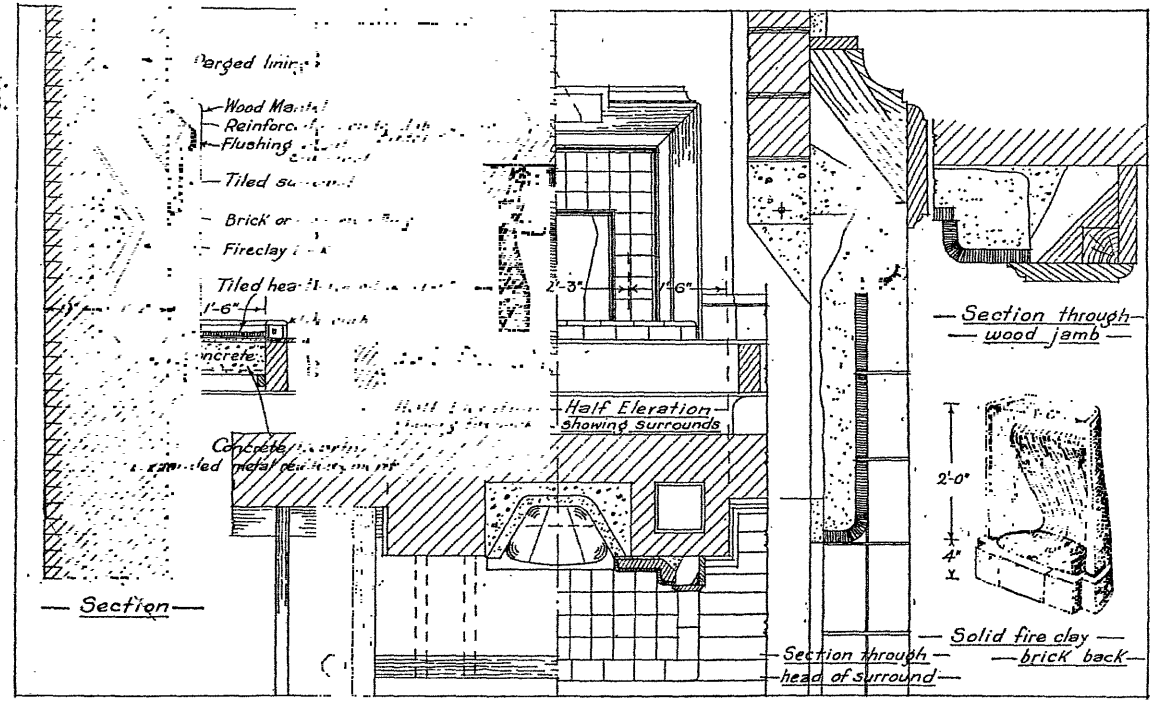


Fig. 212.

Fig. 210. Fig. 211. Fig. 213.

Figs. 208—213.

addition, as the back becomes red hot it tends to more completely burn the soot that rises from the ignited fuel (see Figs. 208 to 210).

Thickness of Backs of Chimneys.—Every flue of a furnace, steam boiler, or close fire used for trade purposes, or of any cooking apparatus or range of an hotel, tavern, or eating-house, must be surrounded with brickwork not less than 9 inches thick for 10 feet above the floor level. The back of a chimney opening in a party wall used for a kitchen range must be 9 inches thick for 6 feet above the chimney opening, and such thickness must be continued at the back of the flue ; the backs of all other chimneys from the hearth up to a height of 12 inches above the opening must be at least $4\frac{1}{2}$ inches thick if in an external wall, and 9 inches thick elsewhere than in an external wall.

Support of Chimney Breasts above Opening.—A sufficient arch of brick or stone, or a bar of iron, must be built over the opening of every chimney, and, if the breasts project more than $4\frac{1}{2}$ inches and the jambs be less than $13\frac{1}{2}$ inches, the abutments must be tied in with a wrought-iron chimney bar, 18 inches longer than the opening. In most cases the breasts project 9 inches or more, and, therefore, if the jambs are less than $13\frac{1}{2}$ inches, these chimney bars must be used, one for each brick in the horizontal thickness of the arch. The ends of chimney bars should be caulked, *i.e.*, cut and turned up and down (see Fig. 207).

Width of Jambs.—The Model Bye-Laws require that the jambs must be at least 9 inches wide on each side of chimney opening, and the brickwork surrounding any flue not less than $4\frac{1}{2}$ inches thick.

Thickness of Brickwork on Upper Side of Flues.—Where flues make an angle less than 45 degrees with the horizon, the brickwork of the upper side must be at least 9 inches thick. This is a necessary provision to secure safety from fire, as if the upper side were not very substantial, the ascending heat might be a considerable source of danger.

Least Height of Chimney Stacks above Roofs.—All chimneys must be carried up $4\frac{1}{2}$ inches least thickness to a height of not less than 3 feet above the highest point in

the line of junction with the adjoining roof, flat, or gutter. It is, however, generally advisable to carry the chimney 3 feet above the ridge-line to prevent down draught.

Maximum Height of Chimneys.—The Model Bye-Laws fix the following as the height to which chimneys may be carried above the roofs in exposed situations, although it may be necessary to make them of somewhat less height :— They shall not be built higher above the adjoining roof, flat, or gutter, than six times the least width of the shaft at the highest point of junction, unless built with and bonded to another chimney shaft not in the same line with it, or otherwise made secure. This does not apply to factory chimneys, for which a special construction is necessary.

Fastenings in Walls of Flues.—No iron holdfast or metal fastenings may be driven nearer the inside of a flue than 2 inches.

Proximity of Woodwork to Flues.—The provisions as to the proximity of woodwork to flues are briefly as follows :—

No woodwork must be built into walls or chimney breasts nearer than 9 inches from the inside of any flue, or under any chimney opening within 10 inches from the upper surface of the hearth. This relates to the insertion of timber beneath the back-hearth or chimney opening, and not, as might be supposed, to the lathing and bearers of the ceiling beneath the front hearth.

No wooden plug may be driven into any wall or chimney breast nearer than 6 inches to the inside of any flue or chimney opening.

No woodwork must be nearer the face of the brickwork or stonework about any flue or chimney opening than 2 inches if the brickwork be less than 9 inches thick, unless the brickwork be properly rendered as before described.

Openings into Flues.—Openings for ventilating valves into smoke flues must not be less than 9 inches from woodwork, in practice such openings are usually placed about 12 inches below the ceiling-line.

Soot Doors.—If any flue be inclined at a less angle than 45 degrees to the horizon a soot door must be provided for

cleaning having an area of not less than 40 square inches and fitted in a proper frame.

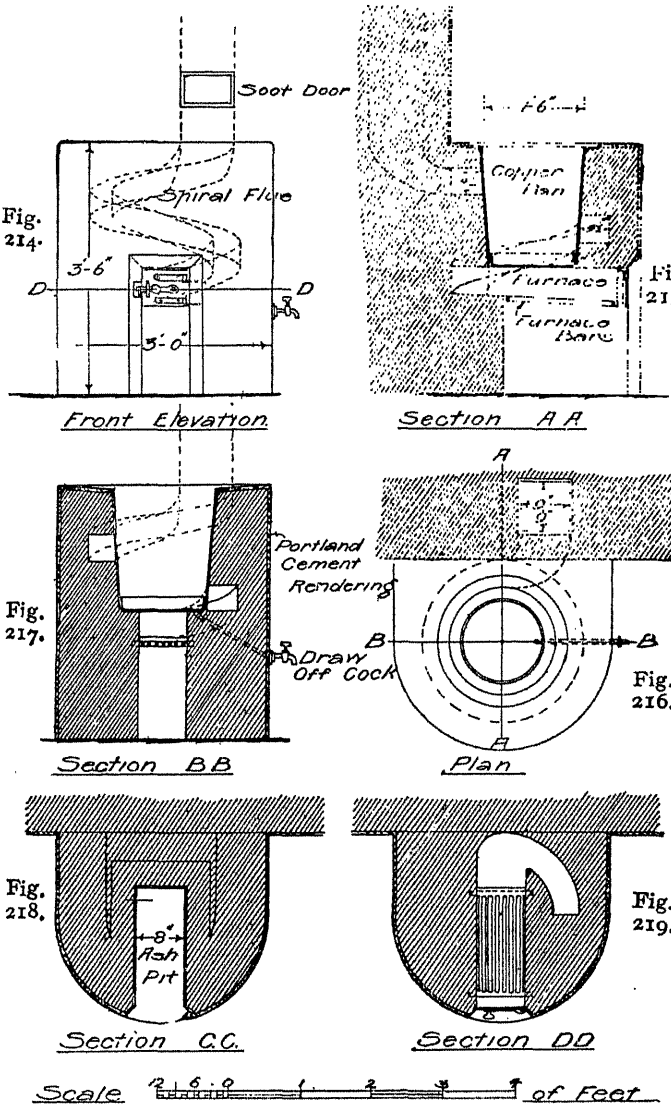
Proximity of Smoke Pipes to Woodwork.—No pipe used for conveying smoke or the products of combustion may be fixed at a distance less than 9 inches from any woodwork.

Copper.—Figs. 214 to 219 show the construction of an ordinary domestic copper. In these arrangements large quantities of water are required to be heated with rapidity for the purposes of cleansing linen and for other domestic purposes. It is necessary that the copper pan should have a large heating surface. A furnace is constructed beneath the pan and with a spiral flue about it. Section C-C shows the outline of the lowest course, section D-D shows the section through the furnace, Figs. 215 and 216 show vertical section through the copper, and Figs. 214 and 217 give the elevation and plan. The brickwork is externally rendered in cement to form a finish. A draw-off cock is shown to remove all water from copper pan when the latter is not in use. These draw-off cocks are frequently omitted, but there is then great difficulty in removing water from the copper pan, especially when the latter is hot, should a change of water be necessary. A soot door should be provided in the lowest part of the straight flue to facilitate the sweeping of the flue. This arrangement is very efficient while it is in working order, but the tendency for soot to accumulate in the spiral flue considerably reduces the efficiency.

Coppers are now very frequently made, with the exception of the pan, entirely of iron. The pan is removable, thus facilitating the cleansing of the flues. The products of combustion are discharged through a branch into a brick flue. These occupy a relatively small space.

In districts where gas is available, coppers made entirely of iron, and heated with gas, have now supplanted the brick-built structure. They are more economical in working, less expensive in first cost, and occupy less space than the above.

Range.—Figs. 220 to 226 show the arrangements of the parts of a kitchen range of modern construction, and


Soot Door
Spiral Flue
3'-6"
D
D
3'-0"
Front Elevation
Fig. 214.
1'-6"
Copper Flan
Furnace
Furnace Bars
Section A A
Fig. 215.
Portland Cement Rendering
Draw Off Cock
Section B B
Fig. 217.
A
A
B
B
Plan
Fig. 216.
8" Ash Pit
Section C C.
Fig. 218.
Section D D
Fig. 219.
Scale 0 1 2 3 4 of Feet

Figs. 214—219.

the setting. An oven and arched high pressure boiler is shown. Economy of fuel is provided for by having a small furnace and causing the heat from the latter to traverse as large a surface of the oven and boiler as possible before finally being conducted up the flue. In this arrangement there are two primary flues, one each for the boiler and the oven. Should the oven be required to be heated rapidly, the boiler damper is closed and the oven damper opened; the heat then passes in the direction indicated by arrows, over the top of the oven, down the side, it is caused to pass towards the front of the oven by a thin iron baffle marked L in the figure, and finally up the flue at the back of the oven. Thus four sides of the oven are directly exposed to the heat. The side adjacent to the fire is protected by a fire-brick. The heat in passing over the top of the oven renders the whole of the hot-plate available for cooking purposes. Should it be required to heat the boiler rapidly the damper is withdrawn, the heat then being induced to pass under and up the back of the boiler.

The boiler is frequently placed at the side of the furnace similarly to the oven, the arrangements for heating the boiler being then identical with that of the oven. The oven and boiler flues discharge into a common flue above the cover plate.

If an open fire is required, the portion of the hot-plate marked Y-Y directly above the furnace is pushed back in the direction indicated by the arrows beneath the portion of the hot-plate in its rear. The flap marked "hinged canopy" is then raised, and two side wings withdrawn; the heat then passes directly up the boiler flue. The furnace bars revolve upon a pivot at their back edge so that their bottom may be raised or lowered to adjust the quantity required at any time; the bottom is kept in any desired position by a rack and pawl arrangement as shown in Fig. 220. For the efficient working of these ranges it is essential that the flues be kept thoroughly clean; for this purpose soot doors are arranged at every change of direction in the flues about the oven and boiler. Figs. 222 and 220 show plan and elevation of range complete; Fig. 224 shows the arrangement of the brickwork seating for the boiler and oven; Fig. 221 shows a similar sketch with

the range front and hot-plate removed, and showing oven, boiler and furnace bars in position; Fig. 223 shows horizontal section through the furnace, oven and boiler; and Figs. 225 and 226 show vertical sections, one through the oven, the other through the furnace and boiler.

For ordinary residential work in districts where gas or electricity is available, the kitchen range as described is nearly obsolete, except for very large establishments, restaurants, etc. With gas and electric cookers, the temperatures are under control; they are cleaner in operation, and the labour in refuelling and the cleaning of the flues is saved.

Hot Water Heating Boiler.

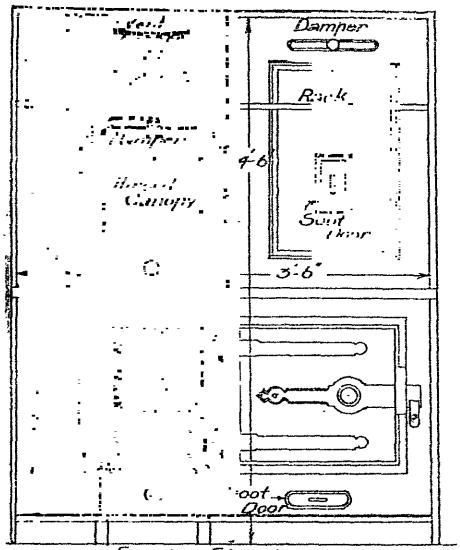
Figs. 227 to 230 show a hot water heating boiler and the method of setting.

In all forms of boilers the principle observed is to cause the flames and heated gases to traverse as much of the boiler surface as possible. The actual capacity of the boiler is small compared with the amount of surface exposed to the heated gases of combustion, thus it becomes rapidly heated. Fig. 229 is a vertical section through the boiler and shows the furnace bars and the ignited fuel; the arrows indicate the direction of the heated gases, first to the back of the boiler, secondly through the central flue to the front, and finally over the top and down the sides of the boiler and up the flue. The transverse section and plan show the form of the boiler and its arrangement relative to the flue, and Fig. 227 shows the front elevation with the soot doors, fire-box and ash pit.

In setting hot water boilers they should be slightly tilted in order that any air may rise always to the one point; at this highest point the flow pipe is fixed, care being taken that the latter does not project into the boiler, to form any air traps. The pipe on its return is taken into the lower part of the boiler.

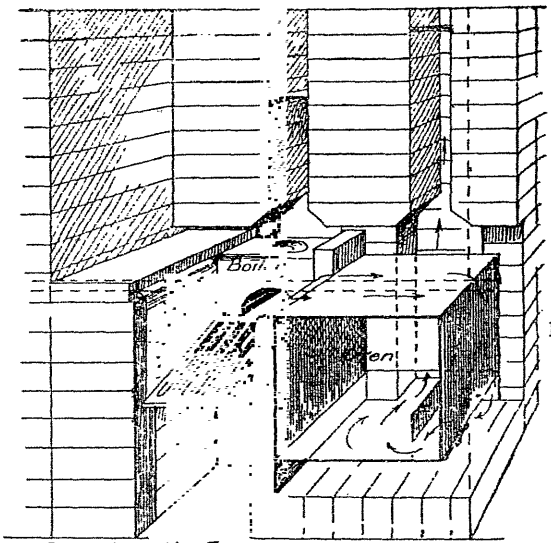
Figs. 227 to 230 show the arrangement of the brickwork about the boiler; the sides of the brickwork in contact with the heated gases should be of fire-bricks.

Small boilers of this capacity are now more frequently formed either of cast- or wrought-iron upright independent



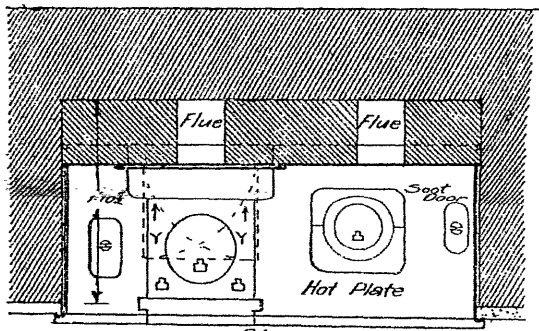
Front Elevation

Fig. 220.



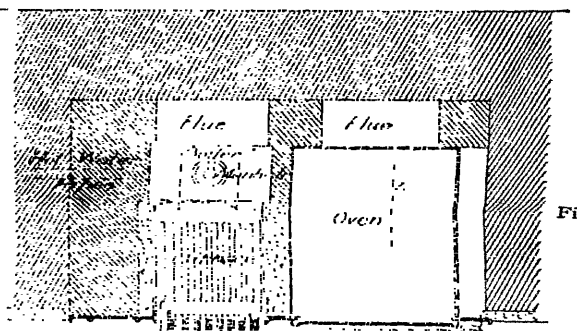
Sketch with Front and Hot Plate removed

Fig. 221.



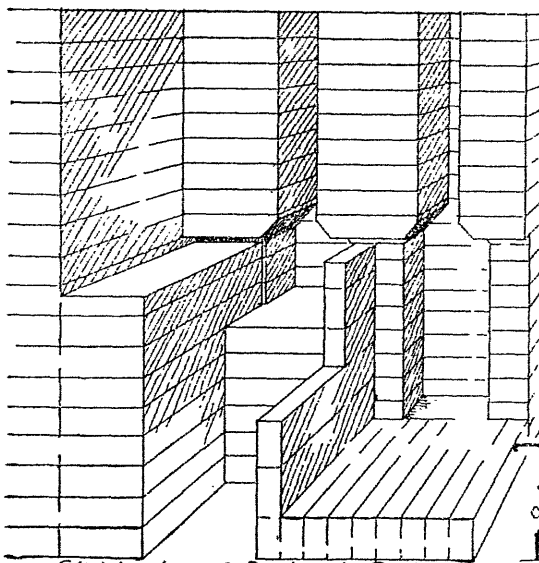
Plan

Fig. 222.



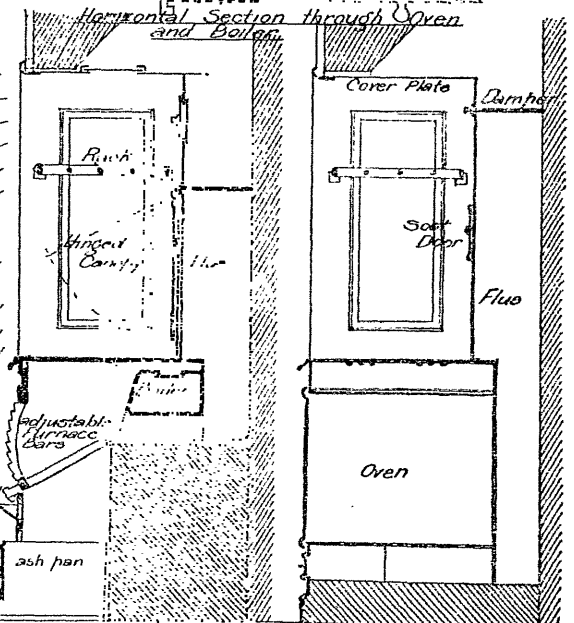
Horizontal Section through Oven and Boiler

Fig. 223.



Sketch showing Brickwork Seating for Range

Fig. 224.



Section through Furnace and Boiler

Fig. 225.

Section through Oven

Fig. 226.

Figs. 220—226.

[Between pages 362 and 363.

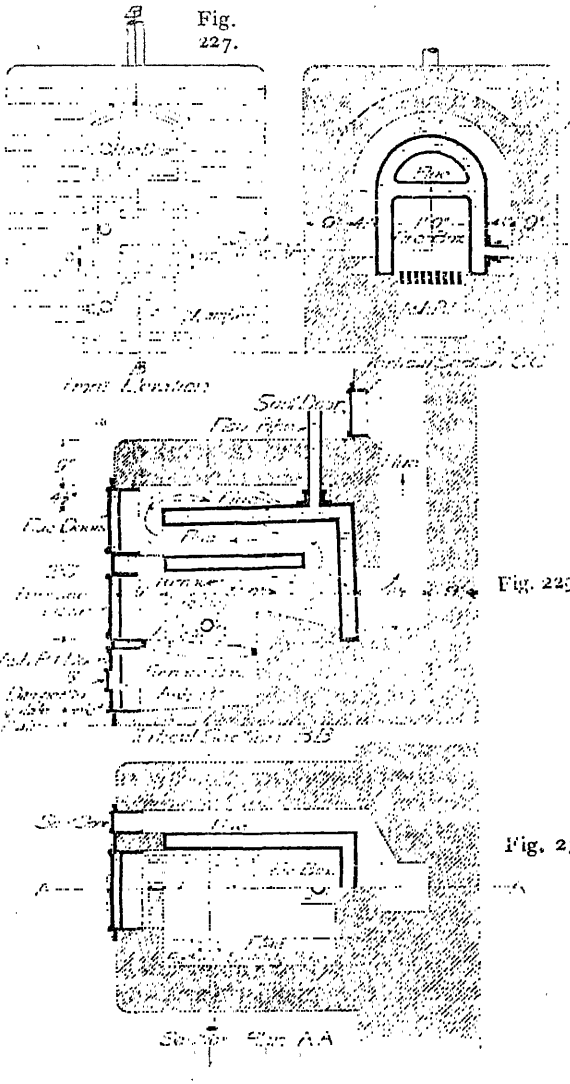


Fig. 227.

Fig. 228.

Fig. 229.

Fig. 230

Figs. 227—230.

boilers, or if of greater capacity sectional cast boilers designed to give the maximum efficiency. These for any given work occupy much less space, and for all ordinary heating installations present many advantages over the old-fashioned brick-set boiler.

TALL CHIMNEY CONSTRUCTION

In determining the dimensions of chimneys, the height is regulated by the draught required, and often to satisfy sanitary conditions, while the dimensions of the diameter are a question of wind pressure.

The following eight paragraphs are taken from the bye-laws of the London County Council.

124.—A chimney shaft shall be of square, circular or any regular polygonal shape.

125.—A chimney shaft and the footings thereof shall be constructed of suitable brickwork jointed with suitable mortar; and the brickwork enclosing such shaft shall be built with a batter (or inclination inwards) of 1¼ inches at least in every 10 feet of height.

126.—The thickness of the enclosing brickwork at the top of a chimney shaft and for 20 feet below the top shall be at least 8½ inches, and shall be increased at least one half-brick for every additional 20 feet or part thereof measured downward.

127.—Any cap, cornice, pedestal, plinth, string course or other variation from plain brickwork in a chimney shaft shall be provided as additional to the thickness of brickwork, required for compliance with bye-law 126 and shall be of proper construction, stability and security.

128.—Footings of brickwork shall be provided immediately below the base of a chimney shaft, and such footings shall spread all round such base by regular offsets to a projection not less than the thickness of the enclosing brickwork at the base of such shaft.

The space enclosed by such footings shall be filled in solid to the satisfaction of the district surveyor as the work proceeds.

129.—Where metal is used in connection with the construction of a chimney shaft or the footings thereof, proper protection shall be provided to prevent damage to such

metal which, in the opinion of the district surveyor, might affect adversely the stability of such shaft.

130.—For the purposes of this bye-law the height of a chimney shaft shall be measured from the base to the top. Such height shall not exceed ten times the least width of such shaft at the base if such base be square and such height shall not exceed twelve times the external diameter or the least width (respectively) of such shaft at the base if such base be circular or of any regular polygonal shape.

131.—Any internal lining in a chimney shaft shall be provided as additional to and independent of the thickness of the enclosing brickwork, and shall not be bonded with such brickwork.

EXAMPLE.—Figs. 231 to 236 illustrate a tall chimney complying with the regulations. Fig. 231 shows half section and half elevation with the inlet flue and manhole for cleaning purposes, also the fire brick lining. This must be quite separate from the outer casing, with a clear space of at least 2 inches. In many cases bricks are projected from the lining into this clear space to nearly touch the outer wall; these prevent any deformation of the inner lining by guiding the expansion upwards. Foot irons are shown in the half section to admit of inspection and repairs. Fig. 232 shows plan through flue and manhole. Fig. 233 shows vertical section through flue. Figs. 234 and 235 show plan through base and centre of shaft, and Fig. 236 shows enlarged detail of a cast-iron cap and cornice surmounting the whole. The dimension of flue at exit should not exceed the area of the exit of fire-brick lining. Tall chimneys were usually built in hydraulic lime mortar, but in a number of modern examples advantage has been taken of the superior tensile and adhesive resistances of Portland cement mortar.

Caps are often of stone, but of late a great many have been made of cast iron, which is more economical and reliable than stone; the latter after fixing, being subject to developing defects which, unless great skill and judgment are used, are liable to be overlooked.

Cramps when made of iron and used in stone caps oxidize and corrode, and often prove very ineffective, and hence should be of gun-metal.

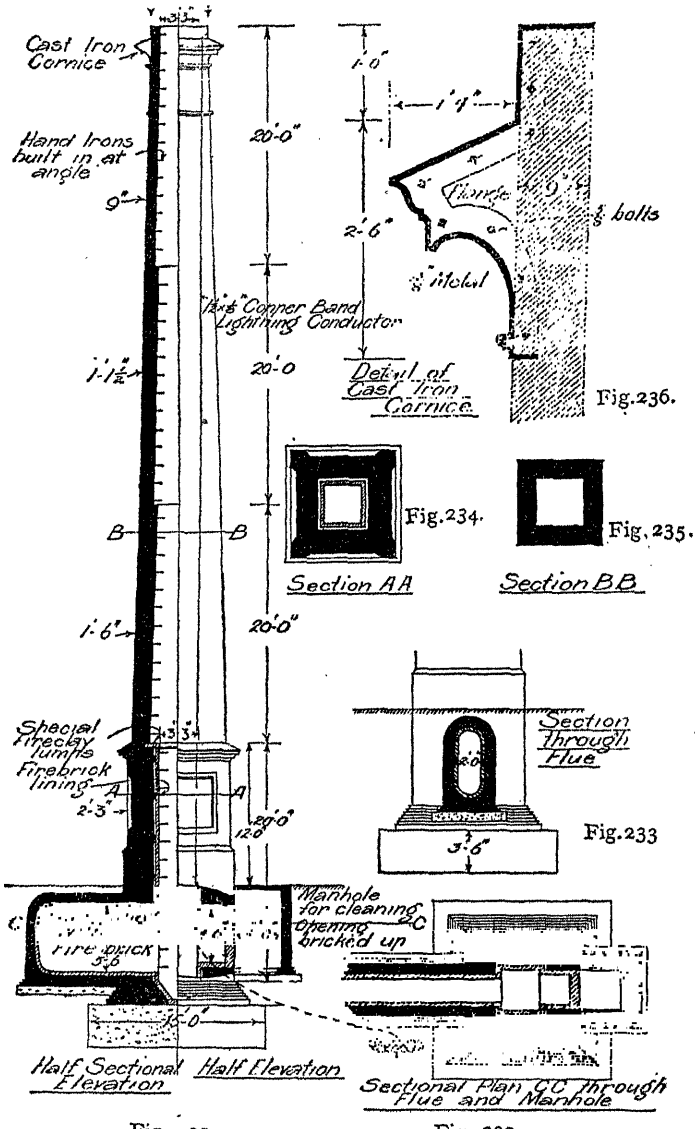


Fig. 231. Fig. 232.

Figs. 231—236.

In circular chimney shafts, where stone caps are used, continuous gun-metal rings are sometimes employed instead of cramps to bind the courses of stonework together.

The limiting position of the centre of pressure of the forces acting upon square and circular chimneys has already been dealt with.

The principal disturbing force acting on chimneys is usually wind pressure, 30 lbs. per square foot being the maximum value usually computed for wind pressure to be resisted by the shaft. This, however, must be determined by local conditions.

In square chimneys, the area exposed to the force of the wind is the height multiplied by the width, that is, the area of its diametral plane.

In circular chimneys the total pressure is the height multiplied by the width of the diametral plane by 0·66 P; the reduced value of the wind pressure is due to the loss by slipping, which amounts to about 33 per cent. Tall chimneys in this country are built in brickwork set in hydraulic lime mortar composed of one of lime to two of sand. It has been thought that cement mortar, though having much greater adhesive and tensile resistances than lime mortar, is subject to deterioration when exposed to the intense heat at the base of a chimney, and fails under a less wind pressure than those built at the base in lime mortar. The success of ferro-concrete chimneys of late years in America and England tends to prove this to be erroneous, and that Portland cement mortar can safely withstand temperatures up to 1,500° Fahr. When this heat is exceeded fire-brick lining set in fire-clay should be used.

There is no direct method of determining the dimensions of a chimney shaft. The usual process is to assume the dimensions of the shaft, and to determine whether it is safe to resist (1) the maximum wind pressure assumed for the locality; (2) to determine if the maximum pressure per square inch on the leeward edge is within the limits of safety for the crushing of brickwork. The data required to determine these is: (1) The weight of the shaft. (2) The position of the greatest eccentricity of the centre of pressure so that the whole of the base under consideration shall be under a compressional stress. (3) The moment of the

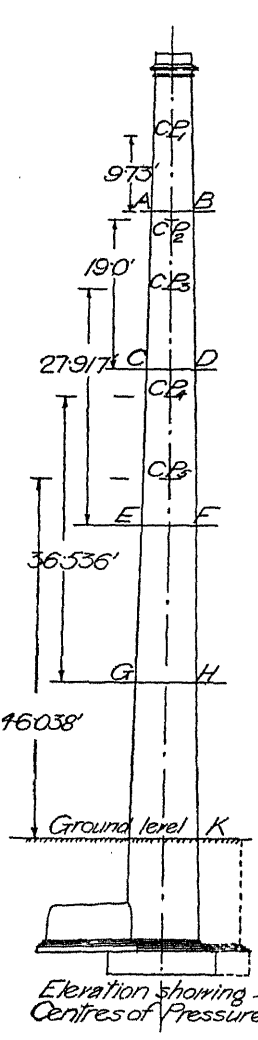


Fig. 237.

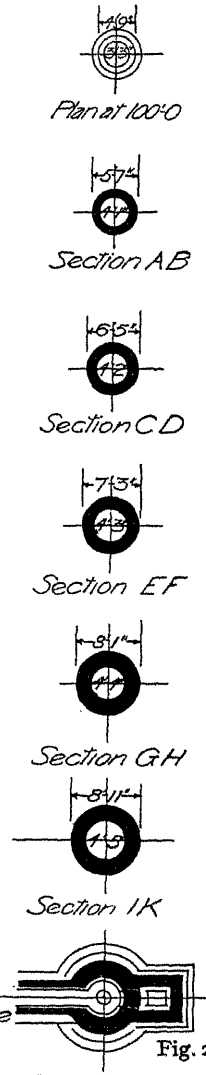
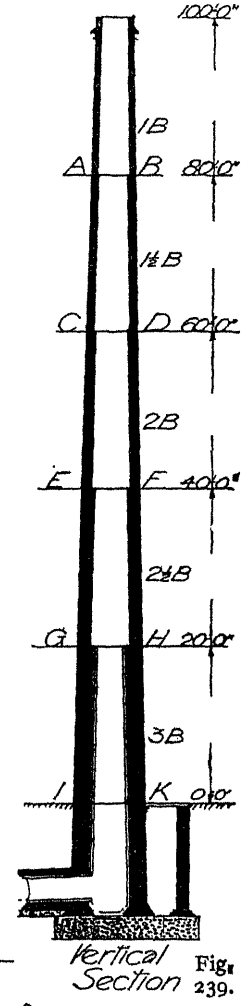


Fig. 238.



Vertical Section

Fig. 239.

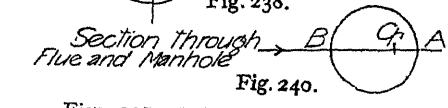


Fig. 240.

Figs. 237—240.

mass of the shaft about the centre of pressure. (4) The height of the centre of wind pressure of the surface exposed to the wind above the base under consideration, *i.e.*, the height of the centre of gravity of the base. (5) The moment of the wind about the base under consideration. (6) To determine the value of the wind pressure per super foot, obtained by equating the moment of the mass of the shaft with the moment of the wind. (7) To determine the normal pressure on the base of the shaft, obtained by dividing the weight of the shaft by the area of the base in inches. (8) To determine the maximum pressure per square inch. This is double the normal pressure when the compression on the base varies from zero on the windward to a maximum on the leeward edge. This data must be obtained for every change of section of the shaft.

The following example (Figs. 237 to 240) for a circular shaft drawn to comply with the regulations of the L.C.C. for tall chimneys is given. The conditions are (1) height 100 feet above the ground; (2) weight of brickwork 112 lbs. per cubic foot; (3) the centre of pressure on the base is calculated for each section according to the method shown on p. 370; (4) diameter of base at ground level 8 ft. 11 in., at top 4 ft. 9 in.; (5) batter 1 in 48; (6) thickness of brickwork 9 inches for top section of 20 feet, and an increase of thickness of $4\frac{1}{2}$ inches for every 20 feet downwards. The assumed area exposed to the wind on a circular shaft is the diametral plane multiplied by a constant 0·66. The reduced area thus obtained compensates for the loss of pressure due to slipping on the sides of a cylinder.

The following symbols are employed :—

m = mass of brickwork above any given level.
D = mean external diameter of shaft between any two sections.
d = mean internal diameter of shaft between any two sections.
D = upper diameter of section of shaft.
D_2 = lower diameter of section of shaft.
h = height of section of shaft.
H = height of centre of wind pressure.
w = weight of brickwork per cubic foot.
e = distance of centre of pressure on base from the axis of shaft.
M_m = moment of mass of brickwork.
M_p = moment of wind pressure.
A = Area of diametral planes.
K = constant of area = ·66.
N = Normal pressure on base per square inch.

Considering the section between the 100-foot and 80-foot levels.

1. Mass of brickwork above 80 feet—

$$m_1 = \frac{\pi}{4}(D + d)(D - d) \times h \times w$$
$$= \cdot 7854\,(8{\cdot}83 \times 1{\cdot}5) \times 20 \times 112$$
$$= 23302 \text{ lbs.}$$

2. Distance of centre of pressure from axis shaft—

$$e = \frac{D^2 + d^2}{8D}$$
$$= \frac{67^2 + 49^2}{8 \times 67}$$
$$= 1{\cdot}072 \text{ ft.}$$

3. Moment of mass about centre of pressure—

$$M_{m_1} = m_1 \times e$$
$$= 23302 \times 1{\cdot}072$$
$$= 25200 \text{ lbs. ft.}$$

4. Height of centre of wind pressure—

$$H = \frac{h}{3}\left(\frac{2D_1 + D_2}{D_1 + D_2}\right)$$
$$= \frac{20}{3}\left(\frac{2 \times 4{\cdot}75 + 5{\cdot}583}{4{\cdot}75 + 5{\cdot}583}\right)$$
$$= 9{\cdot}73 \text{ ft.}$$

5. Moment of wind pressure—

$$M_p = P \times A \times K \times H$$
$$= P \times \frac{4{\cdot}75 + 5{\cdot}583}{2} \times 20 \times {\cdot}66 \times 9{\cdot}73$$
$$= 663{\cdot}5\,P$$

6. Maximum allowable wind pressure—

$$M_p = M_{m_1}$$
$$663{\cdot}5\,P = 25200$$
$$P = \frac{25200}{663{\cdot}5}$$
$$P = 38 \text{ lbs. per square foot.}$$

7. Normal pressure on base—

$$N = \frac{m_1}{\text{area of base}} = \frac{m_1}{\cdot 7854\,(D + d)(D - d)}$$
$$= \frac{23302}{\cdot 7854\,(67 + 49)(67 - 49)}$$
$$= 14{\cdot}2 \text{ lbs. per square inch.}$$

TABULATED DATA OF TALL CHIMNEY (Figs. 237—240)

Section.	Mass of brickwork in lbs. $=m$.	Distance of centre of pressure from axis $=e$.	Moment of mass about centre of pressure.	Height of centre of wind pressure $=H$.	Moment of wind pressure $K=0{\cdot}66$.	Maximum value of P in order that base may be wholly under compression.	Normal pressure on bed joint per sq. in. $=N$.	Maximum pressure on leeward edge $=p$.
100′ 0″—80′ 0″	m_1 23,302 lbs.	1·072 ft.	25,200 lbs. ft.	9·73 ft.	663·5 P	38 lbs./ sq. in.	14·2 lbs./ sq. in.	28·4 lbs./ sq. in.
80′ 0″—60′ 0″	m_1+m_2 23,302+38,595 61,897 lbs.	1·141 ft.	70,600 lbs. ft.	19·0 ft.	2,801·2 P	25·2 lbs./ sq. in.	23·1 lbs./ sq. in.	46·2 lbs./ sq. in.
60′ 0″—40′ 0″	$m_1+m_2+m_3$ 61,897+56,262 118,159 lbs.	1·216 ft.	144,000 lbs. ft.	27·917 ft.	6,633 P	21·7 lbs./ sq. in.	30·3 lbs./ sq. in.	60·6 lbs./ sq. in.
40′ 0″—20′ 0″	$m_1+m_2+m_3+m_4$ 118,159+76,418 194,577 lbs.	1·308 ft.	254,000 lbs. ft.	36·536 ft.	12,378 P	20·5 lbs./ sq. in.	36·9 lbs./ sq. in.	73·8 lbs./ sq. in.
20′ 0″—0′ 00″	$m_1+m_2+m_3+m_4+m_5$ 194,577+110,120 293,677 lbs.	1·388 ft.	407,500 lbs. ft.	44·9 ft.	20,300 P	20·1 lbs./ sq. in.	43 lbs./ sq. in.	86 lbs./ sq. in.
0′ 00″— —13′ 6″	$m_1+m_2+m_3+m_4+m_5+m_6$ 293,677+107,800 401,477 lbs.	1·45 ft.	—	—	—	—	56·7 lbs./ sq. in.	113·4 lbs./ sq. in.

8. Maximum pressure on leeward edge—

$$p = 2N$$
$$= 2 \times 14{\cdot}2$$
$$= 28{\cdot}4 \text{ lbs. per square inch.}$$

From the table on p. 371 it will be seen that the maximum allowable wind pressure over the whole surface is 20·1 lbs., which corresponds to a velocity of about 82 miles per hour, a velocity rarely attained in this country. The maximum pressure on the footings is 113·4 lbs. per square inch, or 7·3 tons per square foot. Reference should be made to the current L.C.C. Bye-Laws for the permissible stresses on brickwork. For stock brickwork in cement 8 tons per square foot is a generally accepted figure, therefore the shaft would be safe from crushing. As the space about the base of the shaft would usually be covered with factory buildings, the wind could not in most cases act with its full force over the lower part of the shaft. If the pressure of the wind should exceed 20 lbs. per square foot over the whole shaft, then the tenacity and adhesion of the mortar would come into play, and would be all on the side of safety.

The above shaft is drawn according to dimensions to comply with L.C.C. regulations, but the area of a shaft is usually determined to satisfy boiler conditions; the diameter may, therefore, be much greater than that given; this would increase the stability and thus, in districts outside the L.C.C. area, make it possible to reduce the quantity of brickwork.

Lightning Conductors.—Chimney shafts and the highest parts of structures elevated above surrounding objects should be protected from the effects of lightning by attaching a metal band of good conductivity to form an electrical connection between the earth and the clouds. All metals are good conductors, compared to the other materials of a structure, and will collect and convey currents, which, if not properly guided away from the building, are likely to form a source of danger to the structure. All metal parts, such as gutters, should be connected with the conductor. The lightning conductor is usually a band or rod of copper or iron commencing at the highest part of the structure, where it is connected to a terminal extending from 4 to

6 feet above the highest part. The end of the terminal is pointed, and has a number of pointed branches extending out from the central stem. The conductor is fixed to the wall, often close to it, and is continued down to the ground; here it is diverted horizontally as far away from the structure as is necessary to find good moist earth, and attached to a metal plate, usually copper, about $\frac{1}{16}$ inch in thickness, and at least 3 feet square, the actual area depending upon the nature of the earth, which should be wet; the drier the ground, the larger therefore will be the required area. In order to obtain an efficient contact, the earth-plate is surrounded by powdered coke about 6 feet below the surface of the ground. Copper is a better conductor than iron in the ratio of 100 to 17, and the relative areas to convey a similar current would, therefore, be in the inverse ratio. Copper is easier to manipulate about the various architectural projections, but iron is better to resist fusion. Care should be taken that all joints are efficiently made, so that the pieces joined are in actual contact. Copper should be riveted and soldered, and the iron screwed or riveted according to section.

The zone protected by a conductor is generally considered to be that space enclosed by a cone, the base of which is twice its height, hence the necessity in long exposed buildings to have conductors at all salient points. Fig. 231 shows the shaft protected by a lightning conductor attached, taken down to the ground and properly earthed.

It has recently been shown that the conductor must be not only a good conductor but as free from electric inertia as possible, avoiding all bends and smoothing down all changes of directions. On similar grounds the single rod is changed for a series of stout wires rising at all corners and making a fringe of points above the roof, the elevation of each point above the roof being very much less than in the case of a single rod. Advantage is taken of all external conductors by making them form a network with the wires provided.

CHAPTER XIV

MASONRY

(Continued from the Author's Elementary Course.)

VAULTING

Classification.—Chambers covered with brick, stone, or concrete are termed vaults and may be classified under three heads :—

1. Barrel Vaults. 2. Domes. 3. Rib and Panel Vaults.

Barrel Vaults.—A barrel vault consists of a continuous arch, resting upon the side walls of a building ; these latter must be very thick in order to resist the thrust of the vault which is distributed along the whole length of the walls ; the latter become in effect continuous buttresses. Where barrel vaults intersect each other the line of intersection is known as a groin. To exactly comply with the statical conditions, every part of the groin should lie in the vertical plane ; for æsthetic reasons also this condition should apply. Such a straight groin can only be obtained when the radii of the intersecting vaults are equal and their springings are at the same level. In all other cases of intersecting vaults twisted groins will result, as shown in Figs. 241 to 244 ; all such intersections in barrel vaults are sources of weakness, and are very apparent. Fig. 244 shows an interior with a barrel vault lit by means of a clerestory formed by smaller barrel vaults intersecting larger. The effect of this, especially where the secondary vaults are large, is to concentrate the pressure of the vaults upon sections of the walls. This is a desirable result, as it enables thinner walls to be built, the points of concentra-

tion only requiring to be fortified by means of cross walls or buttresses.

Figs. 245 to 250 show the plan, sectional elevation and details for the construction of two intersecting barrel

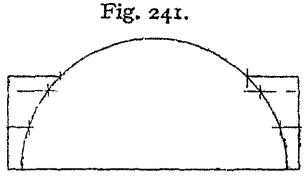


Fig. 241.

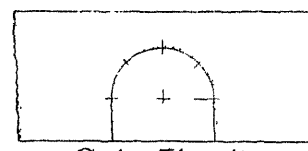


Fig. 242.

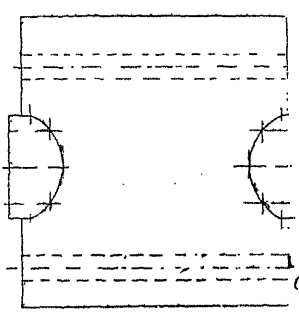


Fig. 243.

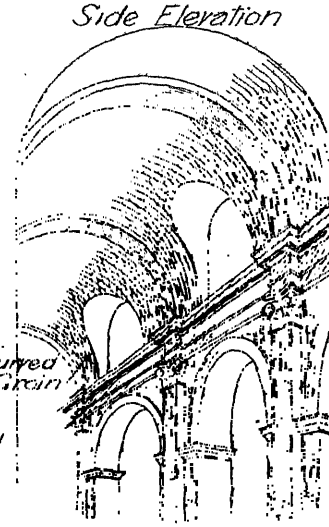


Fig. 244.

Figs. 241—244.

vaults of equal radii. Fig. 246 shows the half plan and intersection of vault looking up. Fig. 248 shows the half plan of the extrados looking down. Fig. 245 is the half-sectional elevation showing intrados and groin. Fig. 247 is a sectional elevation of the vault. Fig. 249 shows the

block of stone out of which No. 3 groin is to be worked, also the templet required. All other groin stones would be projected and worked similarly. Fig. 250 is a perspective view of No. 3 groin when finished.

Domes.—A dome is a roof of the form of a semi-spheroid, ellipsoid, or conoid. In its simplest form it is constructed on a wall circular in plan. The dome is more often supported upon walls square or octagonal in plan, but any other regular polygon would apply equally well. If any polygon is inscribed in the great circle of a sphere and planes perpendicular to the surface of the polygon be projected through its sides, these will intersect the sphere in a number of circular sections. The usual method is to let these circular sections form portions of barrel vaults, as shown in Figs. 251 to 256.

Types of Domes.—There are three distinct methods of arranging the dome: (1) Where the spheroid is intersected by four square walls, as shown in Figs. 251 to 256. In this case the dome appears very flat, a comparatively small portion only projecting above the line of intersection of intercepting walls. (2) In order to obtain greater internal height than in the preceding example let the upper part of the dome above the intersecting walls be considered to be cut off by a horizontal plane resting upon the highest points in the lines of intersection of the walls and dome. This will give a horizontal circle, and upon this a dome of smaller radius than the preceding dome can be constructed, as shown in Figs. 254 and 255. The portions of the large lower dome remaining between the four vertical walls and the horizontal plane are known as pendentives, as shown in Fig. 256; these practically form projecting corbels constructed so that the square or other polygon may be developed into a circle upon which the smaller or true dome may be supported.

In the third case, as shown in Figs. 257 to 259, in order to obtain still greater internal height the smaller dome is elevated upon a circular wall, known as the drum, which rests upon the horizontal circle to which attention has been drawn in the preceding example.

Fig. 245. Fig. 247.

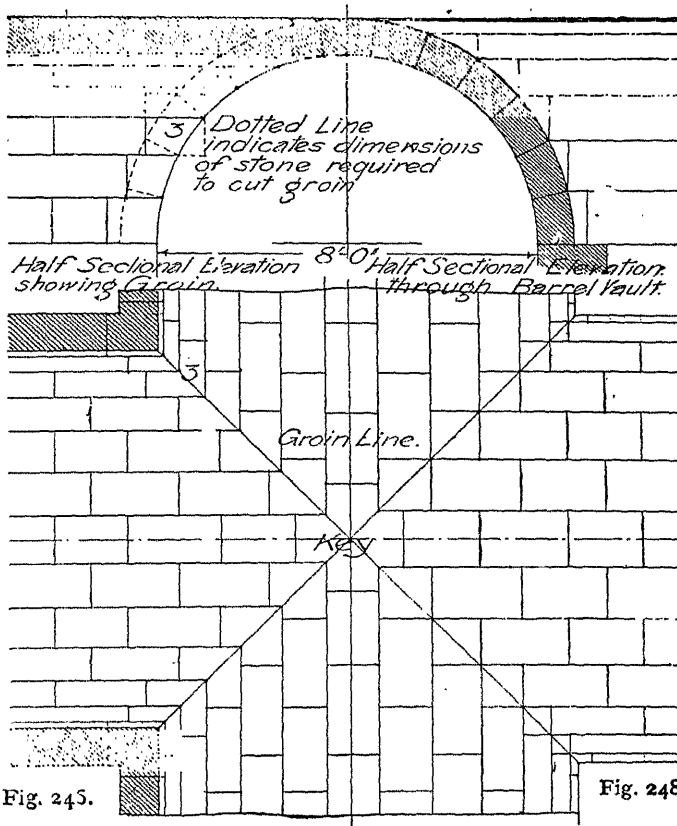


Fig. 245. Fig. 248

Half Plan of Soffit. Half Plan of Extrados.

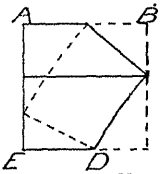
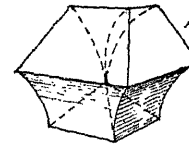


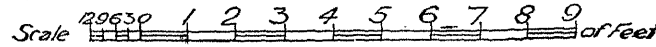
Fig. 249.

Outline of Templet for No 3 Groin.



Perspective view of No 3. Groin Stone

Fig. 250.



Figs. 245—250.

Fig. 251.

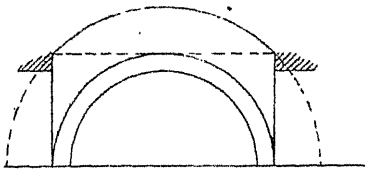


Fig. 252.

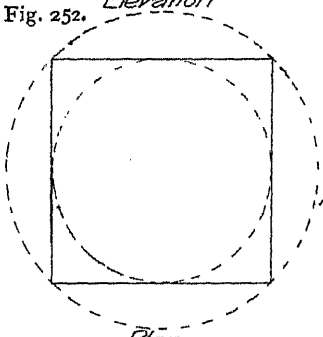
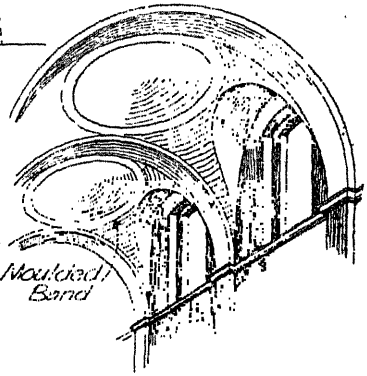


Fig. 253.



General Sketch

Fig. 254.

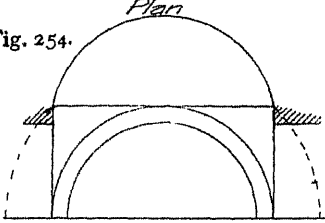


Fig. 255.

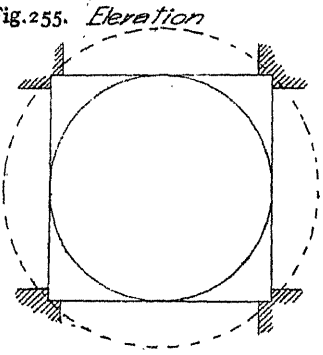
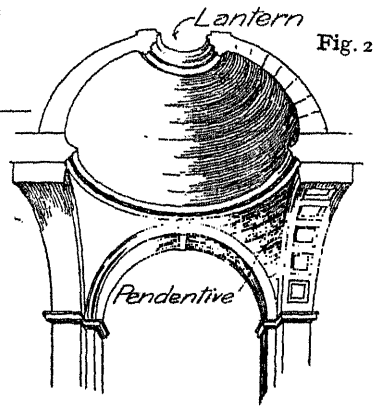


Fig. 256.

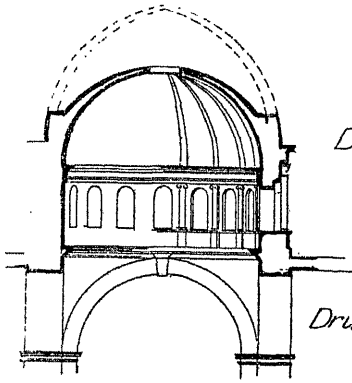


General Sketch.
Pendentive Dome.

Figs. 251—256.

General Description.—Domes are usually constructed of stone or concrete; where of stone they are built in horizontal courses, each of which forms a horizontal arch.

Fig. 257.



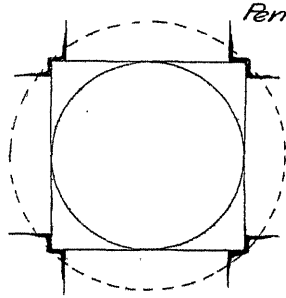


Fig. 258.

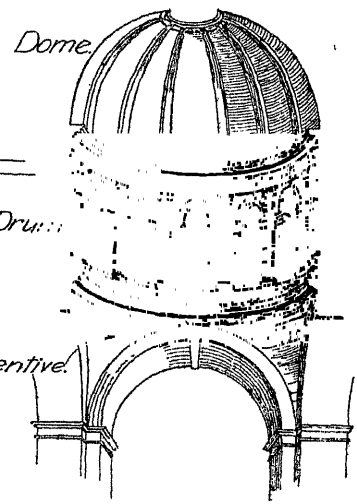


Fig. 259.

Figs. 257—259.

Supposing means to be taken to prevent these rings from spreading or opening, then each ring when complete is maintained in equilibrium by the side thrusts of its several voussoirs and the support it receives from the ring immediately below it. Thus in constructing such a dome no

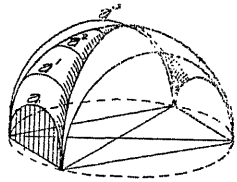
centre is required, only temporary supports until the ring being built is complete; owing to this the central portion may be omitted for purposes of lighting, for which lanterns are usually provided. The lower or upper surfaces of the bed joints of each ring if produced would form a cone. There is a tendency for the dome to spread at a point somewhere between the haunch and the base. In the first method the dome is usually sunk well within the walls and there is no fear of spreading; in the second example the wall may be carried up and constructed of a thickness and weight sufficient to resist its outward thrusts, but in the third case this cannot easily be done without a great expenditure of material and the effacement of the dome as an external feature. To prevent spreading in this case a metal band, encircling the dome at some line between the base and the haunch, is employed to tie the structure together. In many of the large types of domical structures, two domes are employed, an internal and an external dome; this is done to gain effect from both the interior and the exterior. A very tall dome internally presents a cavernous effect and cannot be properly viewed from the inside. A low dome such as would present a good appearance internally would from the exterior appear stunted.

Domes are frequently now constructed of concrete, which has become possible from an economical point of view since the introduction of such a powerful matrix as Portland cement. They are constructed upon a wood centre, upon which the concrete is deposited in regular horizontal rings. If iron ribs be inserted in the concrete, there is a tendency for fractures to occur following the lines of the ironwork. If iron or steel is used it should be in small circular sections interlaced throughout as described in the article on ferro-concrete.

Ferro-concrete domes are now constructed of great magnitude, as in the Wesleyan Memorial Building at Westminster.

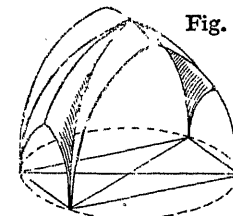
Rib and Panel Vaults.—In the reference to the barrel vaults it was pointed out that the groin was a line of weakness, and where bent was an apparent defect; to remedy which efforts were made to construct vaults of varying

Fig. 260.

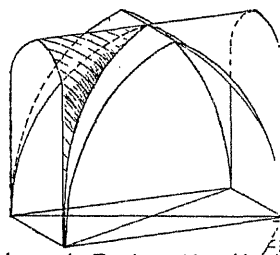


Circular Domical Vault.

Fig. 261.



Pointed Domical Vault



Level Ridge Vault

Fig. 262.

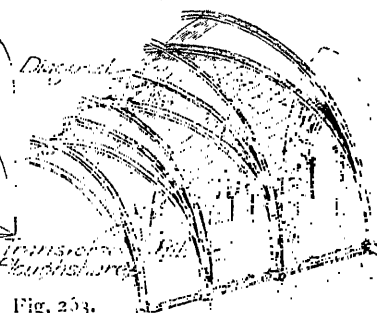
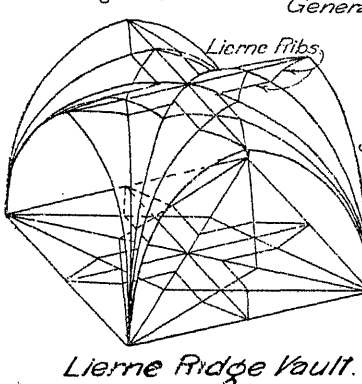
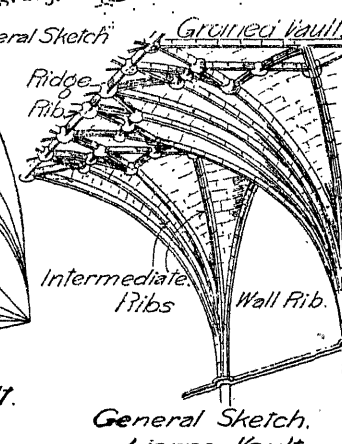


Fig. 263.



Lierne Ridge Vault.

Fig. 264.



General Sketch. Lierne Vault.

Fig. 265.

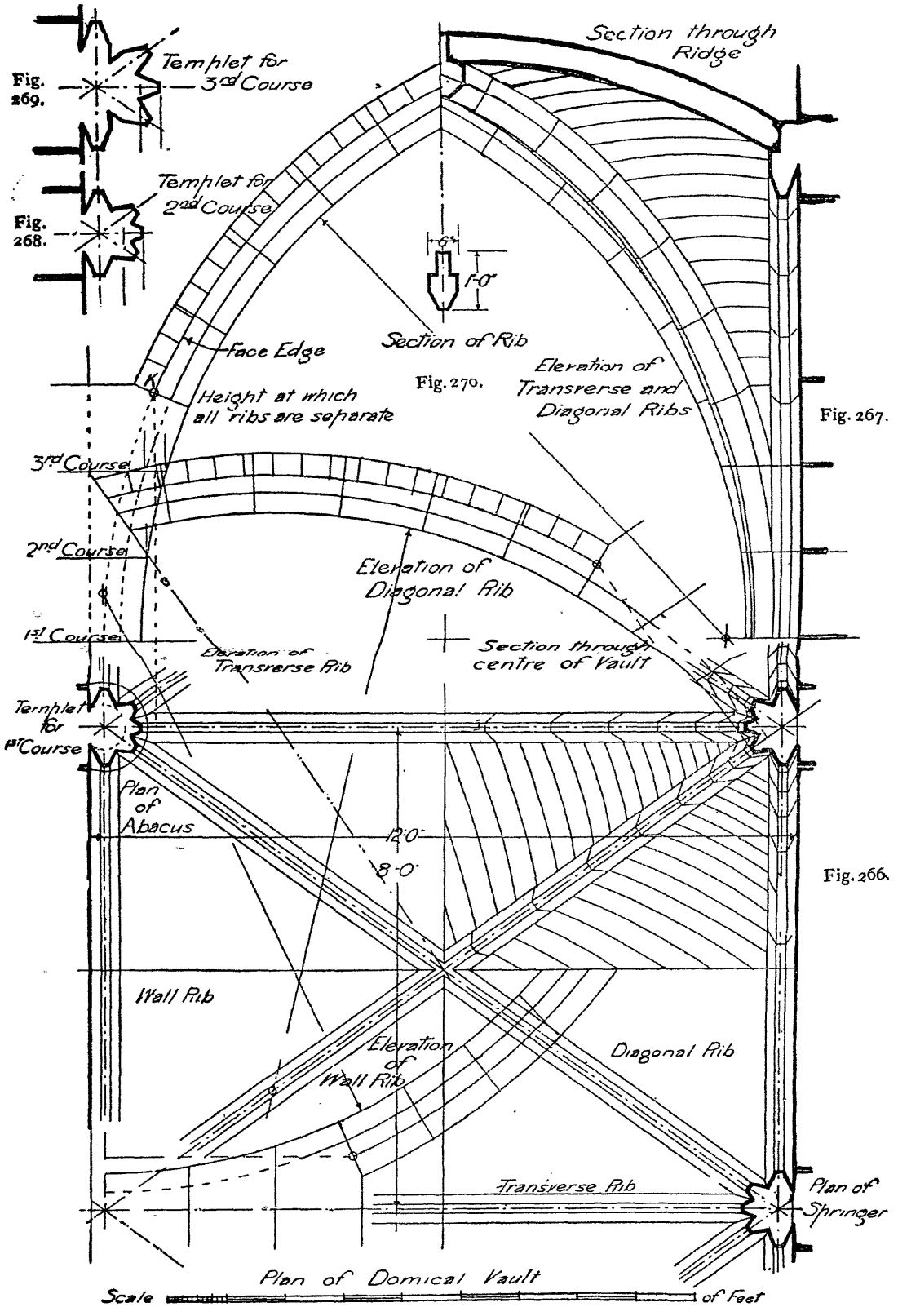
Figs. 260—265.

radii whose line of intersection should be in a vertical plane. If two great semi-circles of a sphere, intersecting at their crowning points, and their extremities are distant from each other an amount equal to the spans of the vaults to be intersected, then let these extremities in plan be joined by lines and upon these latter erect semi-circles. If these semi-circular surfaces be imagined to be moved upward, as shown in Fig. 260, along the great semi-circles, always keeping them in a vertical position, curved surfaces will be generated that will intersect in the great semi-circles as shown in Fig. 260. If these great semi-circles are constructed as two intersecting stone arches they will emphasize the groin, and will form support for the vaulted surfaces. A form of vault or arch that will exert a less horizontal thrust upon the walls or supports is the pointed arch or vault. This form of vault was at first used as a barrel vault, and at a later date for the rib and panel vaults, for the purpose of reducing the horizontal thrust. Fig. 261 shows the form of a pointed groined vault. This and the preceding example are known as domical vaults.

Figs. 266 to 270 show the working drawings for a domical vault. The ribs of this vault, as shown in Figs. 266 and 267, all form segments of similar curves. As all the ribs are of varying spans and segments of similar circles, the apex of each system will occur at different heights, and as the panelling is all built concave on the under surface, the ridge lines joining the apices will be curved also.

In order that the panels may rest upon the ribs uniformly at all points, it is necessary that the face edges, as shown in Fig. 267, of adjacent ribs shall separate at the same height. It will be noted that in an oblong vault, as shown in Fig. 266, the diagonal and transverse ribs intersect at a higher level than the diagonal and wall ribs, but the highest point at which all the ribs separate is taken, and all the ribs constructed as portions of similar arcs up to that point; by so doing all the mouldings and face edges will intersect uniformly. To obtain this height draw the ribs in plan, from the point where the sides intersect erect a projector to cut the face edge in elevation; this will be the required height. At this height, K, the skewbacks in all cases are formed, and if the ribs are of varying curves,





Figs. 266—270.

[Between pages 382 and 383.

the curves commence to differ at this point; up to this height in which all the ribs are combined, the bed joints are made horizontal. Fig. 270 shows the section of the rib employed; this would be cut out of zinc and used as the templet for cutting the arch rib stones; for obtaining the templets for cutting the corbel courses, Figs. 268 and 269, project from the elevation on to the plan of the transverse ribs, Figs. 268 and 269, the increased lengths of the members due to these joints not being normals to the curves. As all the ribs are portions of similar segments, these lines may be swung round from the centre, O, till they cut the centre line of each rib, and thus the projections of each member may be drawn.

The Panelled Surface.—The exact form of the panelled surface is immaterial provided the under surface is concave, the direction of the bed joints may be parallel to the ridge line, as was the common method employed in France; in England it was usual to place the bed joints at some angle to the ribs on which they rested, generally about right angles to the line bisecting the lower angle of the panel; it is important whichever way the courses are laid that they should be concave on their under surfaces, so that each course when complete will form an arch and will only require supporting until that particular course is built, and thus the whole surface will not require a special centre for its construction, as would be required for a barrel vault.

Level Ridge Vaulting.—This principle understood, general efforts were made to raise the crowns of the intersecting vaults to the same level, until finally the ridge lines were level. In the pointed vaults, no matter what the span, the crown can always be raised to any level required by lengthening the radius; but this, where the intersecting vaults vary much in their span, would give an unsightly lancet-shaped slit for the narrow vault. To remedy this the springing of the narrow vault was raised to the height required, thus causing the panelled surface to lie for a portion of its length in the vertical plane, and giving a twisted surface to the panel, termed a ploughshare from its resemblance to that instrument.

Figs. 262 and 263 show a diagram and sketch of a level ridge vault. Figs. 271 and 273 show the working drawings for a level ridge vault. Fig. 271 shows the plan of one bay with the elevation of the diagonal and wall ribs. Fig. 272 shows the elevation of the transverse rib and a half-sectional elevation taken through the ridge of the vault and the position of the ploughshare panel, the diagonal and transverse ribs are struck to the same curve up to the height of the point K, at which point they commence to have a separate existence and from which the arch proper commences ; it may be noted that the springing of the wall rib has been raised, the result of which is to form a ploughshare panel. Fig. 273 shows the front elevation of the ribs at the springing. Fig. 271 shows the method of projecting the joint lines in plan from the elevation of the rib.

The direction of the courses of the filling in or panelling must make an angle with the ridge. The courses are generally arranged at right angles to the line which bisects the lower angle of the panel.

The reason governing and satisfied by this method of procedure is that it is required that each course when laid shall form an arch in itself. If this be done no centres are required for the panelling, but only temporary supports for each course till they are completed.

Lierne Rib Vaulting.—In the later developments of the rib and panel vaulting there was a tendency to cover large spans ; this necessitated the introduction of intermediate ribs between the diagonal and the wall or transverse ribs to strengthen the panelled surfaces. This rendered it imperative to employ ribs at the ridge also. The next stage was to stiffen the groin ribs by means of shorter ribs placed between them ; these were arranged to some geometrical pattern, generally a star-shaped figure, and were termed " lierne ribs " from the French word, " lier " to bind. To facilitate the setting out and to simplify the construction, the centre plane of these ribs was kept vertical ; this rendered it impossible to properly intersect the mouldings at the junction of the ribs, but this difficulty was surmounted by substituting for the intersection a boss of

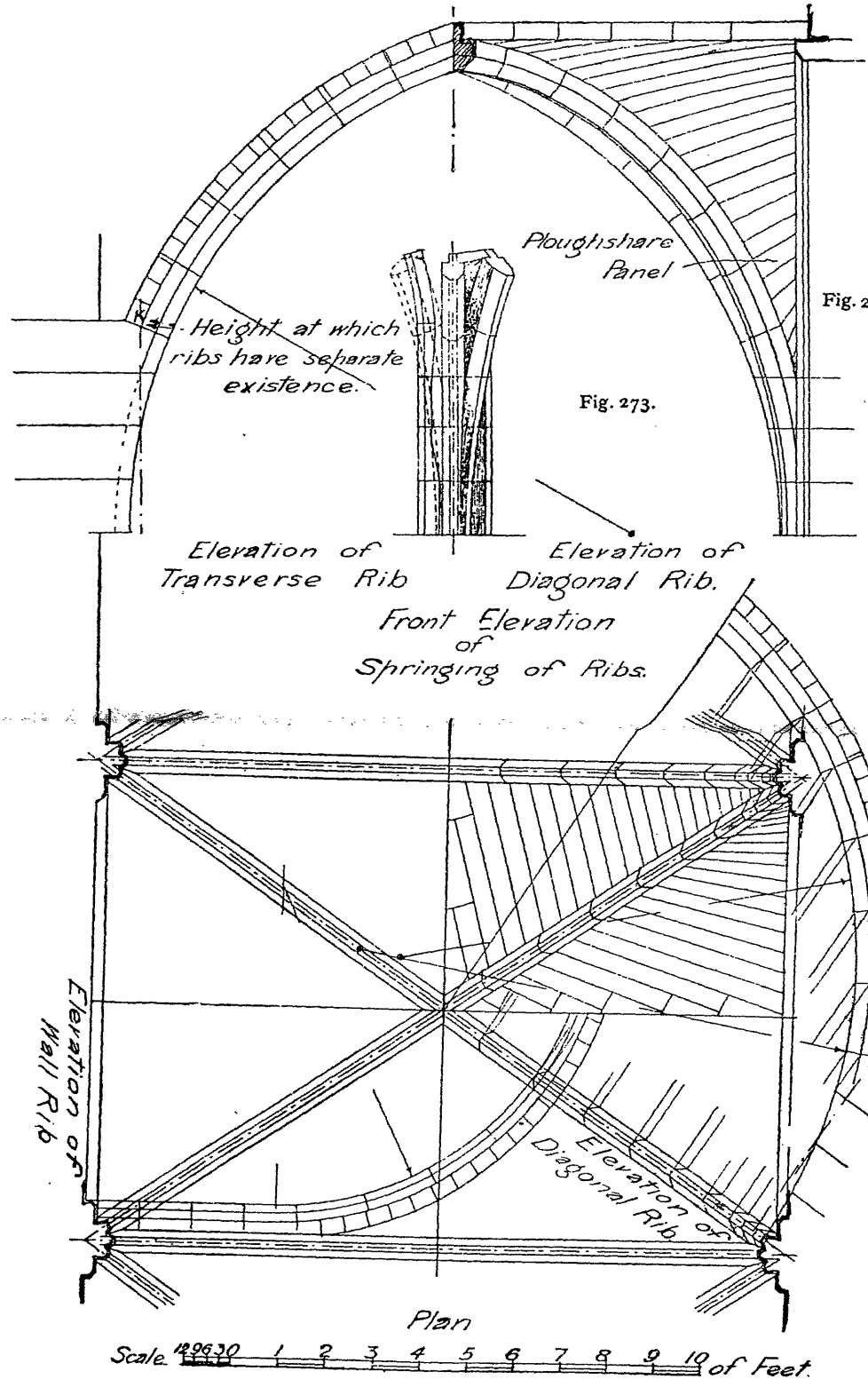


Fig. 272.

Fig. 273.

Fig. 271.

Level Ridge Vaulting.

Figs. 271—273.

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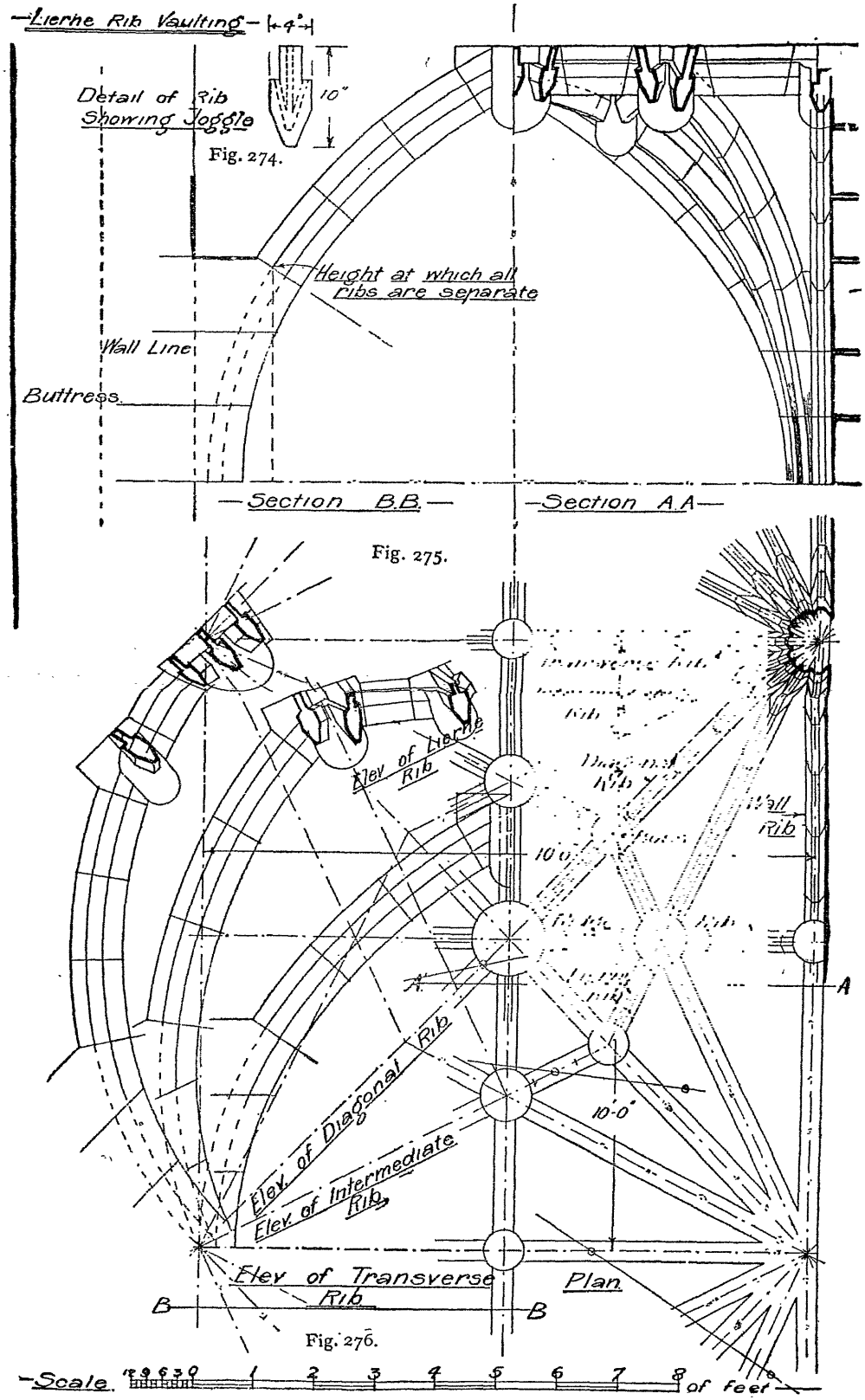


Fig. 274.

Fig. 275.

Fig. 276.

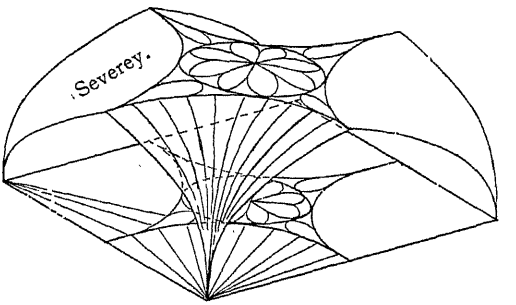
Figs. 274—276.

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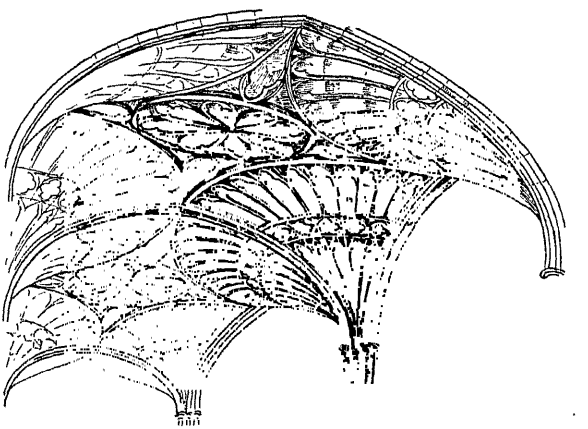
stone, generally carved. Figs. 264 and 265 are diagrams showing lierne ribs and a general sketch.

Figs. 274 to 276 show the working drawings of a lierne

Fig. 277.



Fan Vault.



General Sketch Fan Vault.

Fig. 278.

Figs. 277—278.

rib vault. Fig. 276 gives the plan of the vaulting as seen from below, also elevations and true shapes of the transverse, intermediate, diagonal and lierne ribs. It may be

noted that the boss stones at the points of intersection are formed level on their upper surfaces; on this the angles and widths of the projecting arms can be accurately set out. The bevils for all the bed joints for the boss stone can also be obtained from these projections. Fig. 275 shows two sections, one at B B showing the section through the transverse rib, wall, and buttress; the other, through A A, showing the elevation of the vault. Fig. 274 shows a section of the rib employed, with the position of the cement joggle.

Fan Vaulting.—The tendency of the development of the rib and panel vaults was to increase the number of the intermediate ribs, at the same time to make them lighter, which led up to constructing every rib as a portion of similar curves springing from the same point, and the ribs having an equal angular distance between them which culminated in the severeys becoming portions of inverted conoids. The intersection of the conoids gave an undulating line along which a ridge rib was usually formed. The number of ribs was increased to such an extent that in the fully-developed style they ceased to exist as separate members but became merely projections formed by the sinking of the panels. Horizontal ribs, as shown in Fig. 278, were formed at intervals in the height of the conoid for decorative purposes and to afford an opportunity of increasing the number of ribs as they approached the ridge. In some instances the ridge ribs were continued through from end to end of the chamber; in others the quadrilateral surface in the centre of the compartment left between the intersecting horizontal ribs was filled in by circular panels or other traceried designs having no reference to the ribs of the conoid, as shown in Figs. 277 and 278. The chamber was frequently divided into compartments by transverse arches and each compartment roofed by four inverted quarter conoids.

STONE STAIRS

Stone stairs consist of a number of blocks, fixed at regular and convenient heights, to facilitate transit between

planes of different levels, and are of three kinds : (1) Those stairs supported at both extremities ; (2) those fixed at one end (the other end being left free), and known as hanging steps ; (3) steps circular in plan. These latter are divided into two classes : (1) Those with a central newel ; (2) those with an open well.

The steps may be in one of two forms, either rectangular or spandrel, as shown in Fig. 279. In the commoner stairs the rectangular blocks are used, but where a good appearance is desired, or to gain headroom, spandrel steps are employed. The spandrel steps may be finished in one of three ways : (1) with a plain soffit, which consists in finishing the soffit in one plain surface, as shown in Fig. 279 ; (2) a broken soffit may be employed, as shown in Fig. 279 (this is used for one of three reasons, or for all combined, (*a*) to gain strength at the back of the tread ; (*b*) to save the expense incurred in working the surface of each step perfectly level ; (*c*) to obtain effect ; (3) moulded with a soffit).

Each step may simply rest upon the one below it, but it is usual for the upper step to be rebated over the back of the one below to prevent sliding. To avoid acute angles at this point and to form an abutting surface, particularly in the spandrel steps, a chamfer is taken off the top back edge of the lower step at right angles to the pitch of the stairs, the upper step having a corresponding sinking to fit. This is known as a back joint, and is shown in Fig. 279.

Fixing the Steps.—Stone stairs are erected in one of two ways : (1) They may be built in the walls as the latter are built, or (2) spaces may be left in the walls to receive the ends of the steps, which are fitted and fixed when the wall is finished. The wall should be built in cement mortar, for at least 12 inches above and below the line of the stairs, the gaps to receive the stairs being temporarily filled up by brickwork bedded in sand.

The ends of the steps should be pinned in the walls not less than $4\frac{1}{2}$ inches to 9 inches with tiles or slates set in cement, care being taken that the space left about the end

of the step is filled up, as far as possible, with solid material, leaving no thick mortar joints to squeeze out. While the step is setting, the outer or free end should be supported with wood struts, after being levelled, which should remain until the cement has thoroughly set.

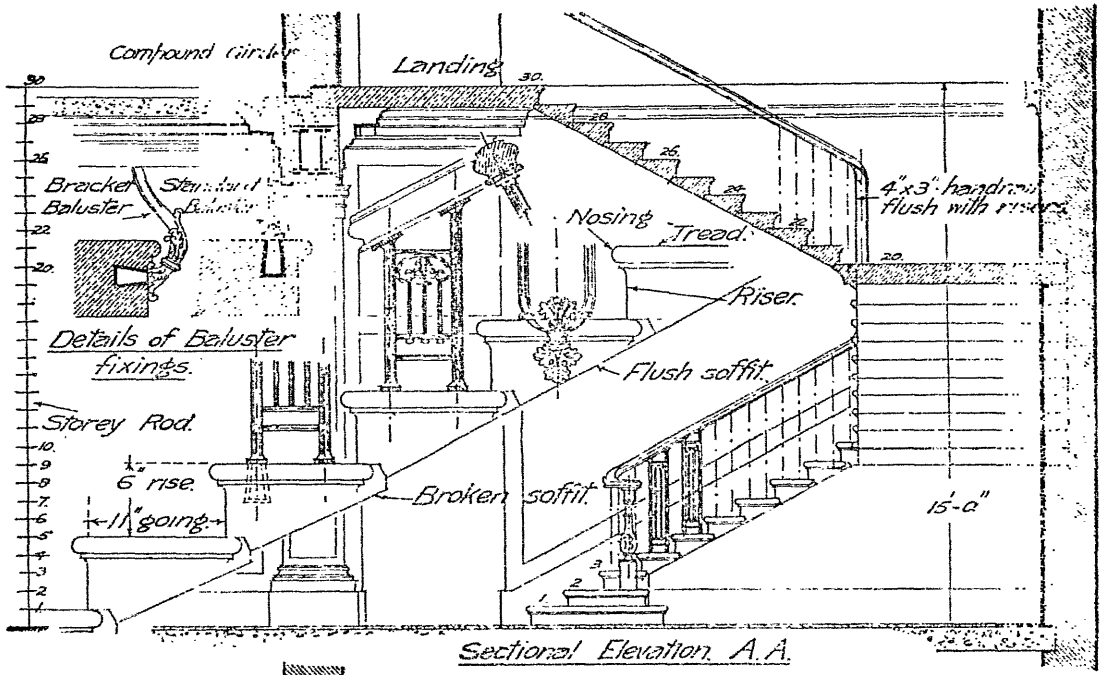
The first kind of stair, viz., those supported at both ends, combine convenience with the greatest strength. They are much used in schools, theatres, and other public buildings. They are usually made of rectangular steps, which rest 6 inches on the wall at either extremity.

The second kind, or hanging steps: these are much superior in appearance to those last described. They derive their chief support from the walls, but each step receives an additional support from the one directly beneath it. The amount of support thus received is indeterminate, and such steps should be supported by steel strings or by arches when they exceed from 5 to 6 feet in length. The actual projection depends upon the stone employed, the value of the resistance to transverse stress having a wide range of values in different stones. These are used for all conditions of stairs, from the secondary staircases in dwelling-houses to the grand staircases in public buildings. In the commoner kinds, rectangular steps are used; but in the superior, spandrel steps are always employed.

The steps may be plain or have moulded nosings; where the latter are employed, the moulding should be returned about the free end, the moulding on the latter being returned and stopped directly beneath the riser of the steps above, as shown in Fig. 279.

Turret Steps.—The first of the third class of stair, the circular newel, is used for turret steps; they are built in a circular chamber. The steps are wedge-shaped, their thin end being worked circular to a radius of about 3 inches, the front edge of each step being tangent to this circle, or radiating from the centre, as shown in Fig. 282, the back edge of the step being a radial line. The steps are built into the walls of the chamber, at their wide ends, each of the circular ends being arranged to fall directly over the

Fig. 279.



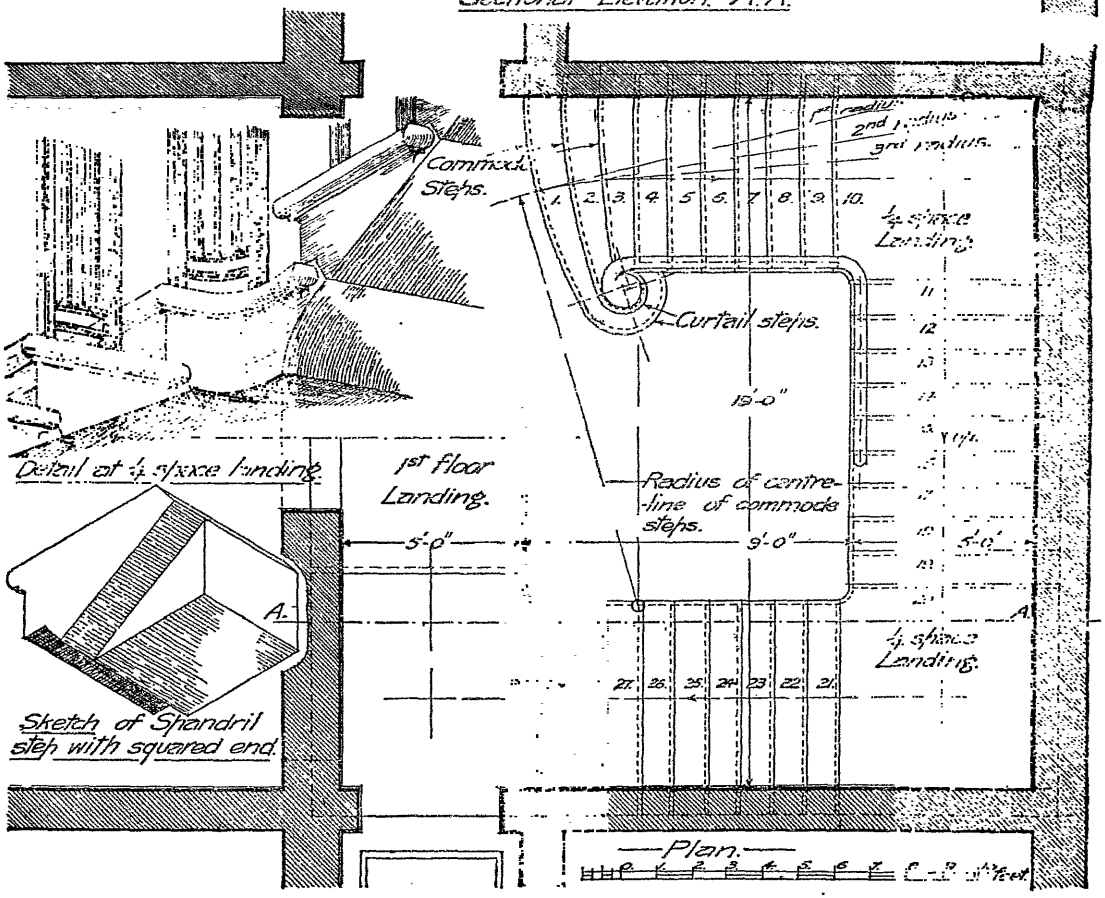


Fig. 280.

Figs. 279—280.

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one beneath it, thus forming a continuous newel up the centre. These form a strong stair, but are rather dangerous,

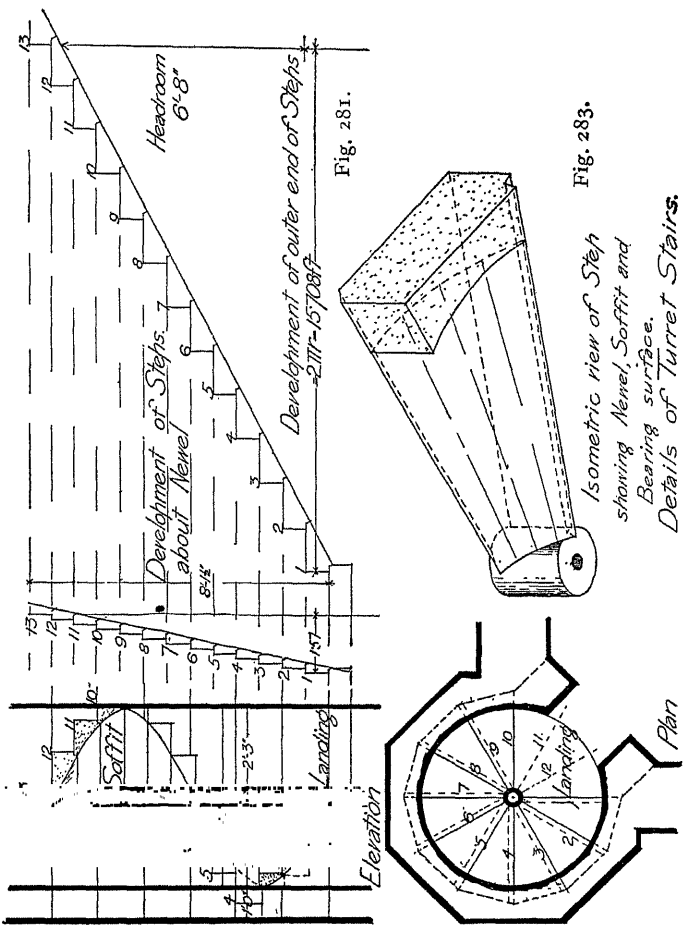


Fig. 281.

Fig. 283.

Fig. 282.

Figs. 281—283.

as they have to be steeply pitched to gain the necessary headroom. Figs. 281 to 283 give the details of a flight of turret steps.

Secondly, those formed with an open well are built in the same manner as the hanging stair, of which they form one variety. Stairs, circular and elliptical in plan, are often built between two walls, as in the first class of stair.

Large stone landings which cannot be obtained out of one piece of stone are joggled at their joints, as shown in the *Elementary Course*, and where the slabs are thin, and are likely to be subjected to heavy traffic, should be supported by steel girders.

Balusters.—The balusters in stone staircases are generally of stone or iron. There are two methods of fixing balusters: (1) Fixing them into the top, suitable for standard balusters, as shown in Fig. 279; (2) fixing them into the side, when they are termed bracket balusters; as shown in Fig. 279. Holes are bored in the steps at the proper intervals, being slightly undercut. The ends of the balusters are indented before being inserted; they may be fixed in with lead, Portland cement, sulphur and sand, or asphalte.

Figs. 279 and 280 show plan, elevation, and details for an open well hanging stair, built of Portland stone. The lower flight shows the handrail supported by standard balusters, the upper portion with bracket balusters.

Commode Steps.—Added importance is given to this type of stairs if a few of the lower steps are made curved on the riser, or as commode steps, in addition to being finished as curtail steps. Fig. 280 shows the first three steps finished in this way. To obtain the best appearance it is essential that the greatest curvature should be given to the first step, and it should gradually diminish or approach the straight in the ascending steps. The method of setting out these steps is as follows (see Fig. 280). Here the fourth riser is straight; the line of this should be produced and a centre point selected about one and a half to twice the length of the steps from the extremity of the step. Draw the centre line of the steps, and where it intersects the fourth riser, describe an arc from the selected centre, on this mark the going of the steps 1, 2 and 3. From these

points draw tangent lines to the curve ; the intersection of these tangents with the wall surface will give the required centres to strike the curve of the riser. The centres are taken on the wall surface, as the direction of the latter should be a normal to the curve of each step. The curtail curve is made a continuation of the curve of the riser.

CHAPTER XV

CARPENTRY

Definition.—Carpentry is the art of framing timber for structural work, and may be divided under two heads—(*a*) temporary, (*b*) permanent.

(*a*) Temporary work includes the timbering for excavations, the erection of scaffolds, gantries, shoring, and the construction of centres and all works raised for the convenience of the workman in building the permanent erection; which are removed on the completion of the building.

(*b*) Permanent work includes all timbering that forms part of the structure, such as wood floors, partitions, half-timbered work, and roofs which remain on the completion of the building.

TEMPORARY WORK

Classification.—The temporary work will be considered under the following heads:—(1) The timbering for excavations, (2) scaffolding, (3) gantries, (4) centering, (5) form work, (6) shoring.

The timbering for excavations has been described in the chapter on Foundations.

Scaffolds

Definition.—Temporary erections, constructed to support a number of platforms at different heights raised for the convenience of workmen, to enable them to get at their work, and to raise the necessary material, are termed scaffolds.

Classification.—Scaffolds are divided into two general kinds: (1) Bricklayers', (2) Masons'.

(1) *Bricklayers' Scaffold.*—The bricklayers' scaffold consists of a number of uprights, called standards, placed about 8 feet apart, these being fir poles about 5 inches in diameter and 30 feet in length. The standards may be increased to any length by lashing a number of poles together; this is done as occasion requires during the erection of the building. The standards rest with their bottom ends on the ground, but to increase their stability and prevent lateral motion the ends are often embedded for about 2 feet in the ground; if any difficulty exists in doing this, a barrel filled with earth is employed to receive the end of the pole, and a York stone flag immediately beneath the standard advantageously extends the bearing surface; they are placed approximately 8 feet apart. Similar poles, called ledgers, placed horizontally and with a vertical distance apart of 5 feet, are lashed to the building side of the standards, 5 feet being the greatest height that the average man can work with ease.

These form a frame, which is erected about 4 ft. 6 in. from the face of the intended building, to which it is connected by means of horizontal members called putlogs, which take a bearing on the wall at one end, and at the other on the ledgers, to which some are lashed, these being wedged to the wall when the wall has been built sufficiently to allow of this being done.

The putlogs are of square timber, usually birch, 3″ × 3″ and 5 feet in length, the pieces not being cut, but split, to ensure the length fibres being uncut.

The putlogs are placed about 4 feet apart, and on them the scaffold boards are laid to form the platform. The boards are 12 feet in length, 9″ × 1½″; the ends are bound in hoop iron, to prevent their splitting.

The scaffold boards at their heading joints are butted, two putlogs being placed at this part about 4 inches apart to support the ends. About the edges of the staging guard-boards are placed, consisting of boards placed on edge and nailed to the standards, as shown in Fig. 284, to prevent material falling.

The City of London Bye-Laws require the lowest stage to be double planked, and guard-boards about the edges, for the purpose above stated. This staging remains until

the scaffolding is taken down, but all the above stages are raised as the height of the building increases.

The frames are braced to add stiffness and to prevent the scaffold rocking, these braces consisting of poles lashed to the outside of the frames to triangulate the latter, as shown in Fig. 284.

For 9-inch walls a scaffold is only required on one side of the walls, but for all walls of a greater thickness a scaffold is required on both sides.

(2) *Masons' Scaffold.*—Masons' scaffolds are constructed on principles similar to the bricklayers', but owing to the increased weight of the materials handled, the whole erection is made much stronger, the standards being placed closer together longitudinally and more firmly braced. Two frames are erected, one on each face of the wall and about 4 ft. 6 in. distant from it (see Figs. 285 and 286). These frames are erected preparatory to the building of the wall; they are connected together by short poles, called cross ledgers, lashed to the longitudinal ledgers, and they are also braced transversely as well as on the face of the frames. These are frequently termed independent scaffolds, as they obtain no portion of their support from the wall. At each staging ledgers are lashed to the cross ledgers on both sides of and parallel to and about 9 inches from the face of the wall. These serve as the inner supports for the putlogs (see Fig. 286), as it would be inconvenient in an ashlar faced wall to fix the putlogs as in a brick wall. The inner ledger is further stiffened by short upright poles or pieces of quarterings resting upon the ledger beneath. As the wall is erected the cross braces are removed and it is frequently necessary to remove some of the cross ledgers, but the greater number should be arranged to pass through window openings to prevent the necessity for their removal (see Fig. 285).

In England it is usual to have scaffolds on both sides of walls for masons' work, but in Scotland, to save the expense of scaffolding, it is usual for both bricklayers and masons to work from the inside of the wall only, the materials being hoisted by means of derrick cranes. The walls are erected and the floors constructed, the latter forming platforms at

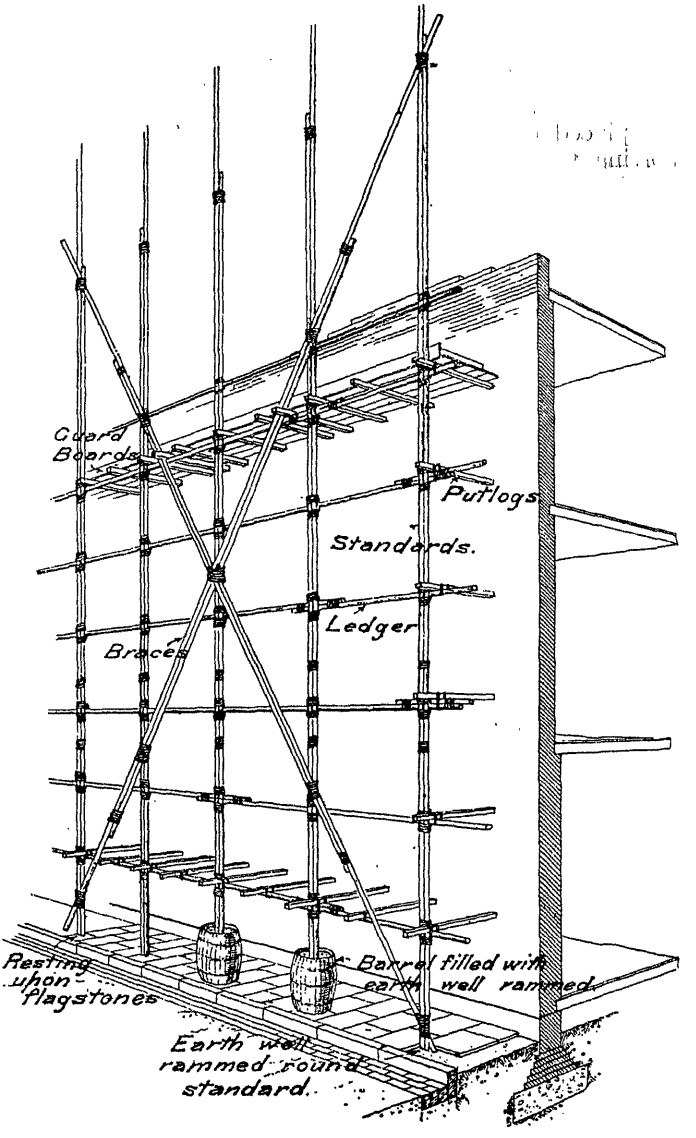

Guard Boards
Putlogs
Standards.
Ledger
Braces
Resting upon Flagstones
Barrel filled with earth well rammed.
Earth well rammed round standard.

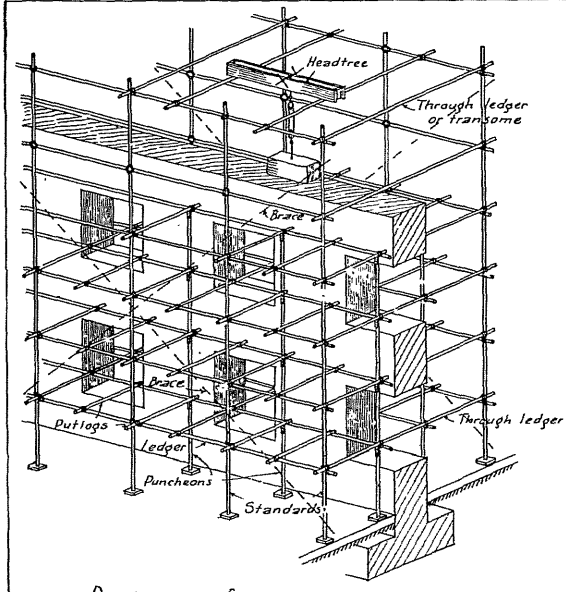
Fig. 284.

these levels ; any intermediate levels are constructed from platforms supported by trestles of about 5 feet in height resting on the floors. If more than one such tier or platform is required between each floor, a second series of trestles are placed on the platform immediately above the supporting trestles. In the case of large cornices, or work requiring special care in setting, a temporary face scaffold is usually projected on cantilevers. The method of building from internal scaffolds only is objected to in the south of England, as it does not admit of the work being so conveniently inspected.

Fir poles for scaffolding are now being rapidly supplanted by steel tubing, which presents many advantages over the older material. The tubing employed is 1½ inches internal diameter steam steel tubes, No. 6 gauge, 3½ lbs./foot run. The standard length of the unit is 18 feet. All kinds of shorter lengths are available, and, if desired, longer lengths than 18 feet can be obtained. The weight of a standard length is 63 lbs. ; the external diameter is 1$\frac{29}{32}$ inches, it can thus be easily handled by the average man ; this, compared with the ordinary 30-feet fir poles, with an average weight of 100 lbs., is an important advantage.

The small diameter and the standard lengths simplify the storage and transport aspect. The safe resistance to crushing of a tube when used as a standard is approximately 3 tons, and if not greater than timber, is more certain ; if overloaded it does not suddenly break like timber, but gives ample warning by bending. Its adaptability to any purpose required on a building job, such as storage racks for timber or any other material, the framing for temporary buildings, sheds, etc., constitutes a valuable asset.

Equal in importance to the tubes are the couplings. These are of three types : (1) the coupling for securing tubes at right angles to each other (Fig. 289), (2) the swivel coupling (Figs. 290 and 291), which allows the tubes to be secured at any angle to each other—these are known as double couplings—and (3) the putlog coupler (Fig. 292), are various types of couplings on the market. Those shown in Figs. 289 to 292 perfectly fulfil every requirement. Firstly there are no loose parts, even the bolts, by a patent device,



— Diagram of —
— Masons' Independent Scaffold —

Fig. 285

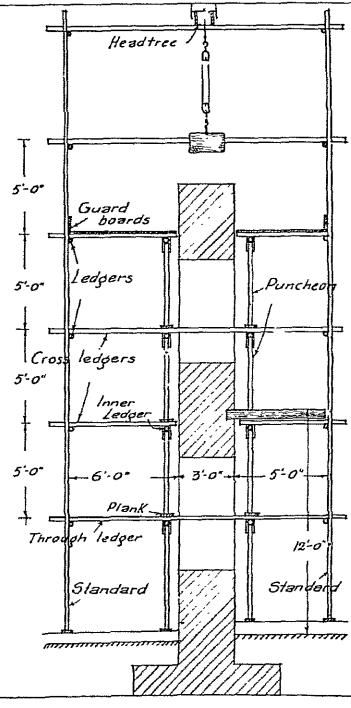


Fig. 286.

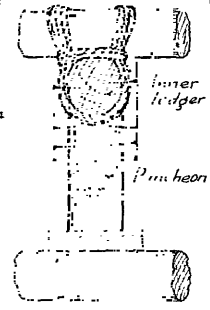


Fig. 288.

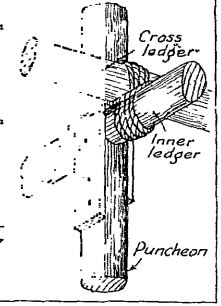


Fig. 287.

Figs. 285—288.

are not detachable. The two parts of the coupling are hinged, and can be easily applied to the standard and ledger, and when bolted grip the tube with a pressure of over 4 tons. The tubes when coupled are fixed rigidly at a dead right angle. The swivel coupling used for bracing can, of course, be fixed at any angle. It is estimated that the connection can be made in a twentieth of the time required for lashing two poles together, and the joints are infinitely more rigid (see Figs. 289 to 292).

The putlog coupler is a simpler attachment than the two former, but like the others, has no loose parts. It is designed to secure two tubes which lie one on the other. It has not got the same frictional grip as the two former types (Fig. 292).

Base plates are provided, 9″ × 9″, with a central pin that fits into the base of the standards. These are sufficient to take the weight of the scaffolds on ordinary firm ground; for soft ground or over cellars or pavement lights stout planking should be provided to distribute the pressure. The plates can be spiked to the planking, holes being provided in the base plates for the purpose (see Fig. 293).

The tubing is in standard lengths, but can be extended to any length, being securely fixed by a joint pin, which is inserted into the ends of the two lengths of tube to be joined, being secured with a coupling joint. A later type has a split pin which opens when the coupling joint is screwed up, and makes a very secure joint (see Fig. 294).

For bricklayers' scaffolds, where the putlogs have a bearing on the wall, the putlog tube may be flattened out and the end driven into the joint. An improvement on this is the putlog head, a simple device consisting of a plate which can be driven or built into the joints of the brickwork; it is provided with a ring coupling to receive the end of the putlog, which is firmly secured by a simple turn of the nut (see Fig. 295).

By the use of this member the practice of leaving putlog holes and the filling in of them as the scaffold is taken down, is rendered unnecessary.

An important and useful accessory is the reveal pin. This consists of a bolt with a circular head at each end, the latter fits the bore of the tube. The bolt is tapped for about

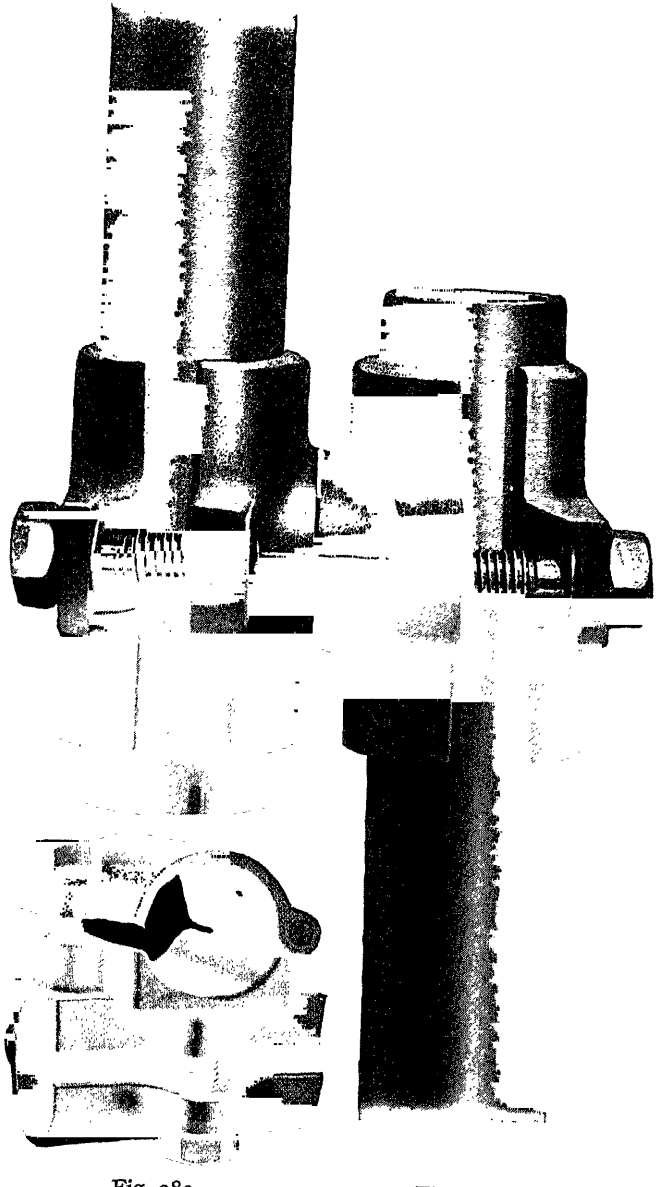


Fig. 289.

Fig. 290.

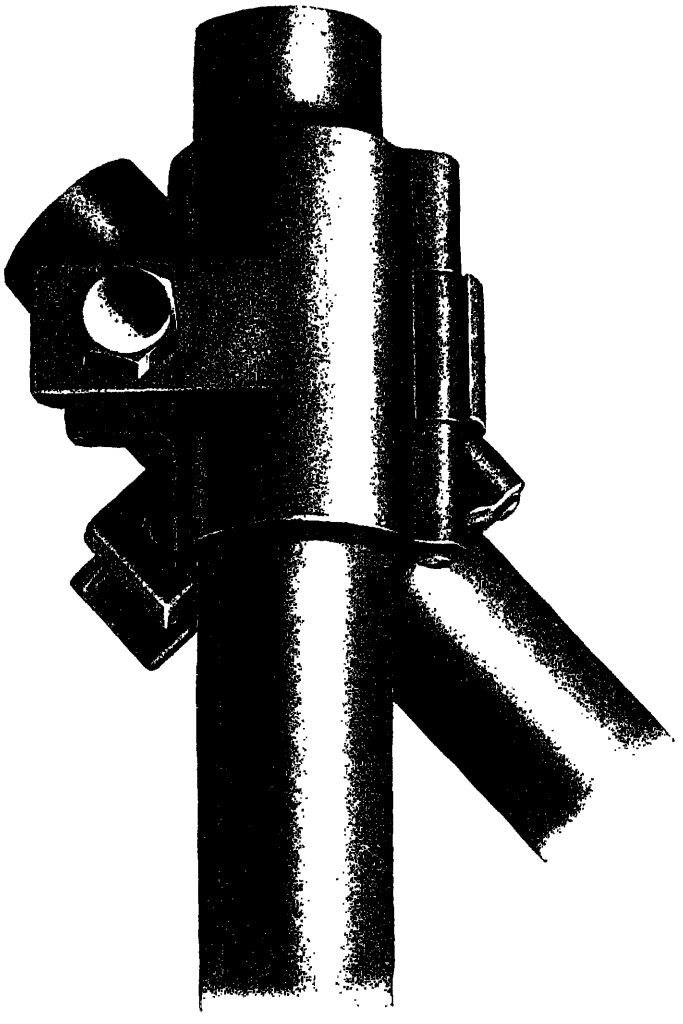


Fig. 291.

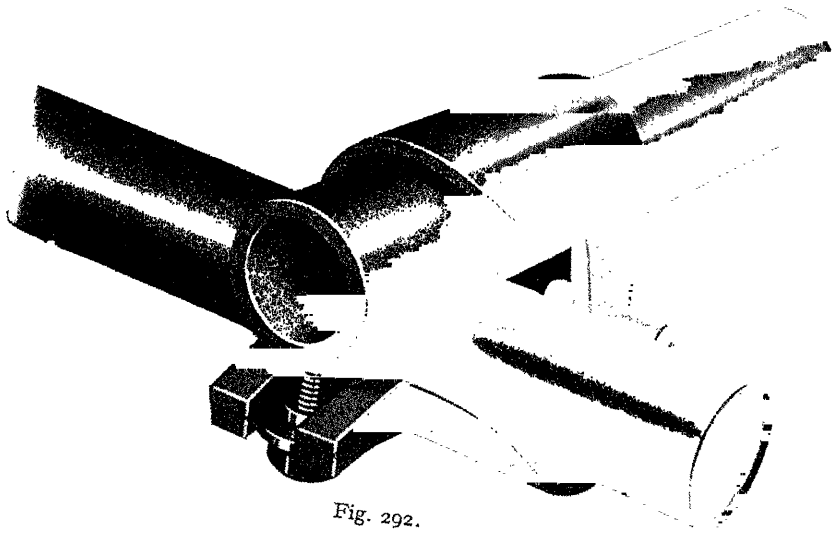


Fig. 292.

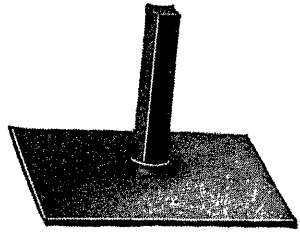


Fig. 293.

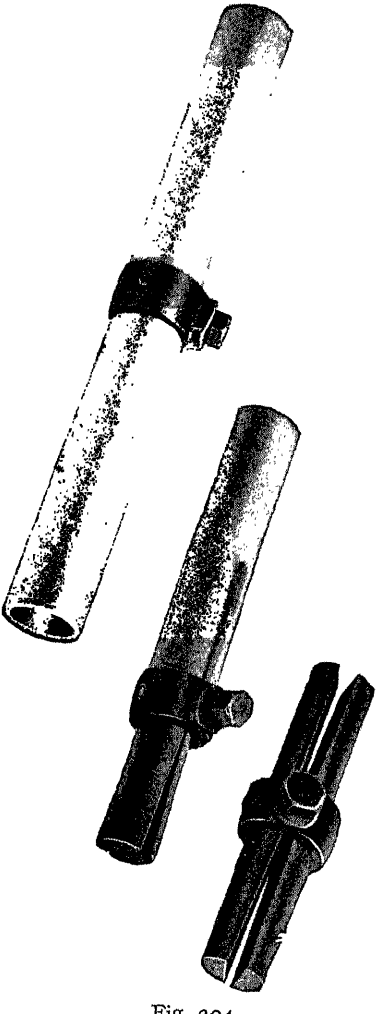


Fig. 294.

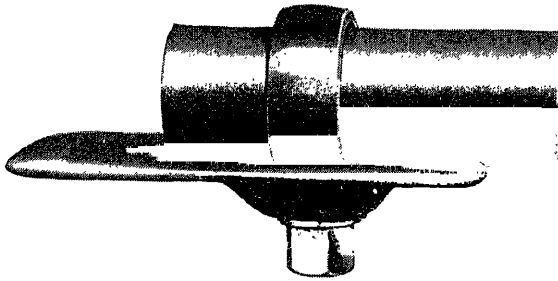


Fig. 295.

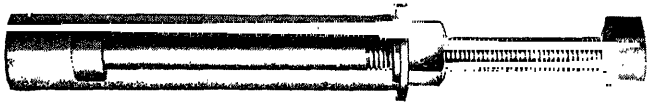


Fig. 296.

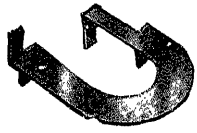


Fig. 297.



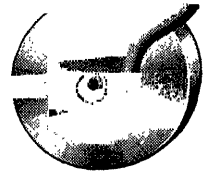


Fig. 298.

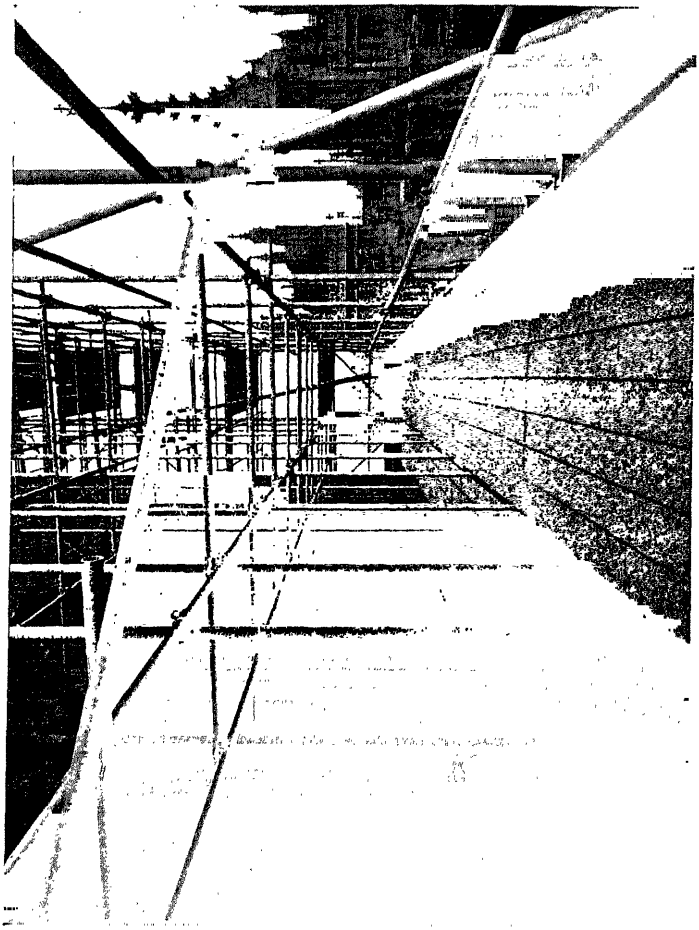


Fig. 299.



Fig. 300.

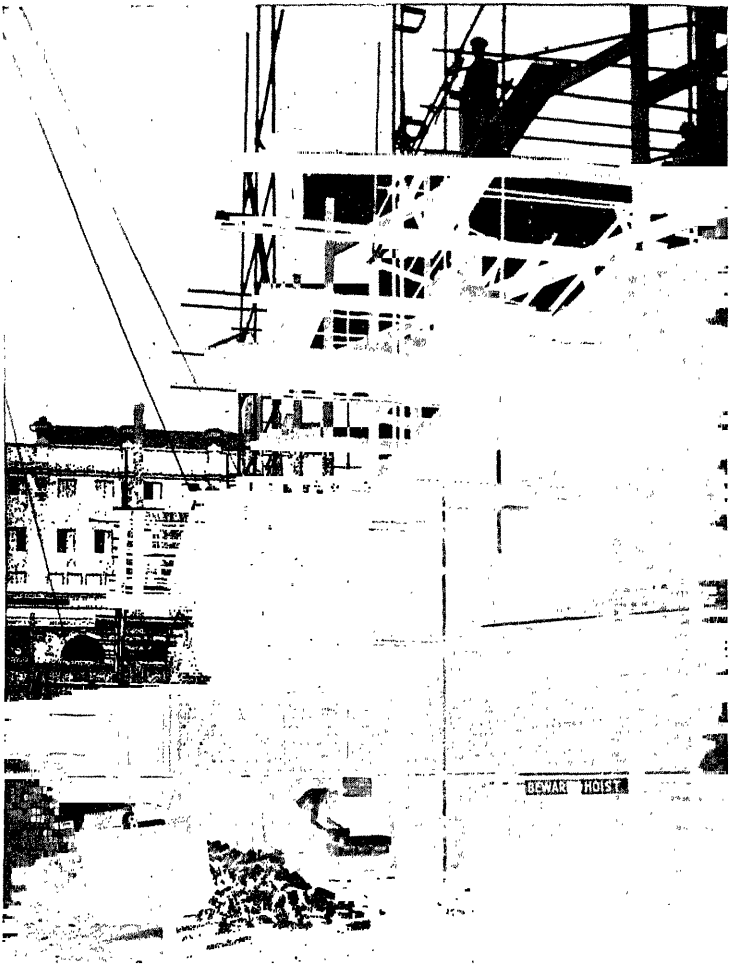


Fig. 301.

half its length, and has a special nut (see Fig. 296). The bolt is passed up any length of tube, this arrangement constitutes a light form of screw jack. When placed between reveals of window or door openings, or between buttresses, it forms a rigid fixing, to which putlogs may be coupled. It can be used either vertically or horizontally. Another useful attachment is the guard board clip. It is employed to secure the ordinary decking to the putlogs or the guard boards to the standards (see Fig. 297).

For internal cleaning or decoration of lofty rooms or halls, movable scaffolds or stagings can be arranged by a castor attachment fitted to the bottoms of the standards (see Fig. 298).

For forming platforms the ordinary wood scaffold boards are being replaced by members formed of sheet steel of a light gauge and made to the size of the standard wood board. To give the requisite strength and stiffness the edges are given a threefold turn, each at right angles, so that the edges present a width equal to the thickness of the wooden boards. The top surfaces of these steel boards are covered with small indentations to prevent slipperiness.

The steel sheeting or wood boards are fixed to the putlogs or, when used as guard boards to the standards, by special steel clips which pass round the tubes and grip adjoining boards tightly (see Fig. 299).

Fig. 299 is an illustration of part of the scaffolding erected for the renovation of the Houses of Parliament, and shows the various types of couplings, the steel scaffold planks, the board clips, and is an excellent example of a masons' scaffold, and shows the adaptability of this type of scaffold to restoration work.

Fig. 300 shows an example specially illustrating the putlog fixing that avoids leaving out a half brick.

Fig. 301 illustrates the method of constructing a hoist for raising bricks, etc., during the erection of the structure.

These figures and data of steel tubular scaffolding have been supplied by the Steel Scaffolding Co. Ltd., of Regent Street.

Government Scaffolding Recommendations.—A memorandum issued to master builders on building accidents

and their prevention, embodies the following suggestions :—

1. All working platforms above the height of 10 feet, taken from the adjacent ground level, should, before employment takes place thereon, be provided throughout their entire length on the outside and at the ends,

(a) with a guard rail fixed at a height of 3 ft. 6 in. above the scaffold boards. Openings may be left for workmen to land from the ladders, and for the landing of material ;

(b) with boards fixed so that their bottom edges are resting on or abutting to the scaffold boards. The boards so fixed should rise above the working platform not less than 7 inches. Openings may be left for the landing of the workmen from the ladders.

2. All "runs" or similar means of communication between different portions of a scaffold or building should be not less than 18 inches wide. If composed of two or more boards they should be fastened together in such a manner as to prevent unequal sagging.

3. Scaffold boards forming part of a working platform should be supported at each end by a putlog, and should not project more than 6 inches beyond it unless lapped by another board, which should rest partly on or over the same putlog and partly upon putlogs other than those upon which the supported board rests.

In such cases where the scaffold boards rest upon brackets the foregoing suggestion should read as if the word bracket replaced the word putlog.

N.B.—Experiments have shown that a board with no more than a 6-inch projection over a putlog can be considered safe from trapping or tilting.

4. All supports to centering should be carried from a solid foundation.

5. In places where the scaffolding has been sublet to a contractor, the employer should satisfy himself, before allowing work to proceed thereon, that the foregoing suggestions have been complied with, and that the material used in the construction of the scaffold is sound.

Definition.—Gantries are structures either temporary or permanent erected primarily to facilitate the loading and unloading and transmission of material, and also for the storage of the same during building operations. They are of three types, differing in form to serve their particular requirements, *i.e.*, (1) the staging erected in front of buildings in course of erection in urban districts, designed to act as unloading platforms. They extend usually from the face of the intended structure to the edge of the curb, covering the footways. A gangway is provided under the staging for the public convenience; part of the footway is usually disturbed, this part is boarded. Hoisting tackle is invariably provided for raising material from carts on to the platform for distribution to the various parts of the job, either in the form of shearlegs, conveyors or travelling cranes, which can run on rails from end to end of the gantry. (2) Gantries erected to support travellers: these are commonly erected in contractors' yards, and consist of two parallel frames. The heads of the frame are provided with rails on which a movable staging, spanning the space between the frames and mounted on wheels, can run from end to end of the gantry frames. This movable staging or traveller consists of two beams on which rails are fixed. On the latter is placed a winch mounted on wheels and capable of moving from end to end of the beams. Thus material can be unloaded and moved to any part of the space between the main frames. (3) Stagings to support derricks.

Landing Stagings.—These are constructed of two frames placed usually from 8 to 10 feet apart, out of square timber, ranging from 6 to 12 inches, according to the work in hand. The frames consist of a head, a sill called a sleeper, uprights and braces. The sleepers are first laid on the ground, the uprights placed in position and dogged to the sleepers, the heads are placed on the uprights and dogged in a similar manner to the sill; where a joint occurs in the head it is made to come over an upright, a short piece of deal being placed on the top end of the latter, to increase the bearing, the whole being dogged together (as in Fig. 302). Braces, usually out of about $4'' \times 4''$, are cut and fixed between

the uprights, the heads of the struts butting against each other or against a straining piece, their bottom ends resting on cleats nailed to the uprights; the two frames are also braced together by having braces spiked or bolted to their standards (as shown in Fig. 302). The platform is formed by laying deals flat, side by side, bearing on the heads of the two frames, or by placing them on edge from 1 to 2 feet apart, and covering them with a flooring of scaffold boards. The inner frame is kept at least 1 foot from the face of the proposed work. If the gantry is over the public way, it must be double planked to prevent dust, rubbish or water falling upon foot-passengers (City of London Bye-Laws).

Figs. 302 and 303 show elevations of a gantry with shearlegs for the hoisting of material, and the commencement of a scaffold placed on the gantry.

Gantries of this type are now being largely constructed of steel. Figs. 304 and 305 show an arrangement of the light steel frames employed. The uprights consist of light steel joists. These are connected by bolting steel channels at their bottom, which act as sleepers. Their upper ends are connected with a light lattice framework, bolted on the face of the pillars and resting on cleats. The two frames are connected with cross frames bolted to them. The various parts are standardized, which ensures simplicity and rapidity in erection, and enables the parts to be used many times. The inner frames of both the timber and the steel gantries frequently have to be taken down to a basement level for a bearing; in this case either longer standards are employed, or a subsidiary frame is erected to support the upper frame, being cleated to the upper lengths of the pillars by fish plates through the web.

Figs. 304 and 305 show the method of placing the scaffold on the gantry, also a conveyor for raising material from the road to the platform, from whence it can be transferred to other parts of the job, either by other hoisting appliances or by suitable runs.

Gantries to support Travellers.—These are formed of two frames, constructed in a similar manner to those described, but as the space between the frames is required to be clear the latter can only be connected at their two

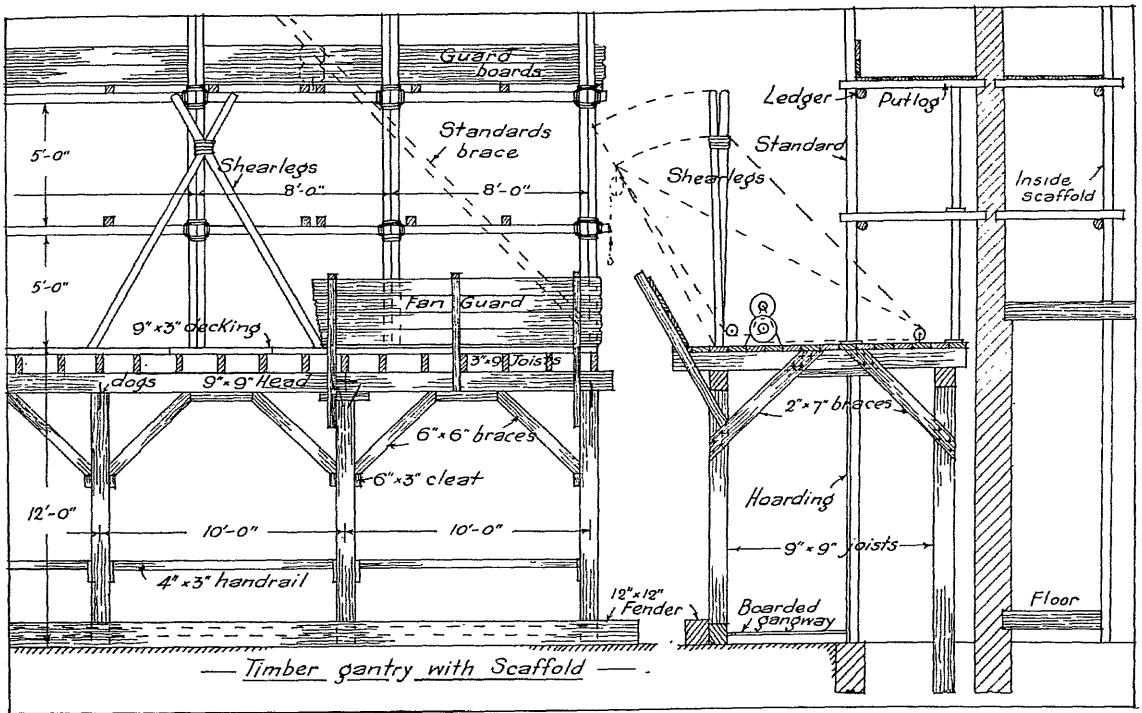


Fig. 302. Fig. 303.

Figs. 302—303.

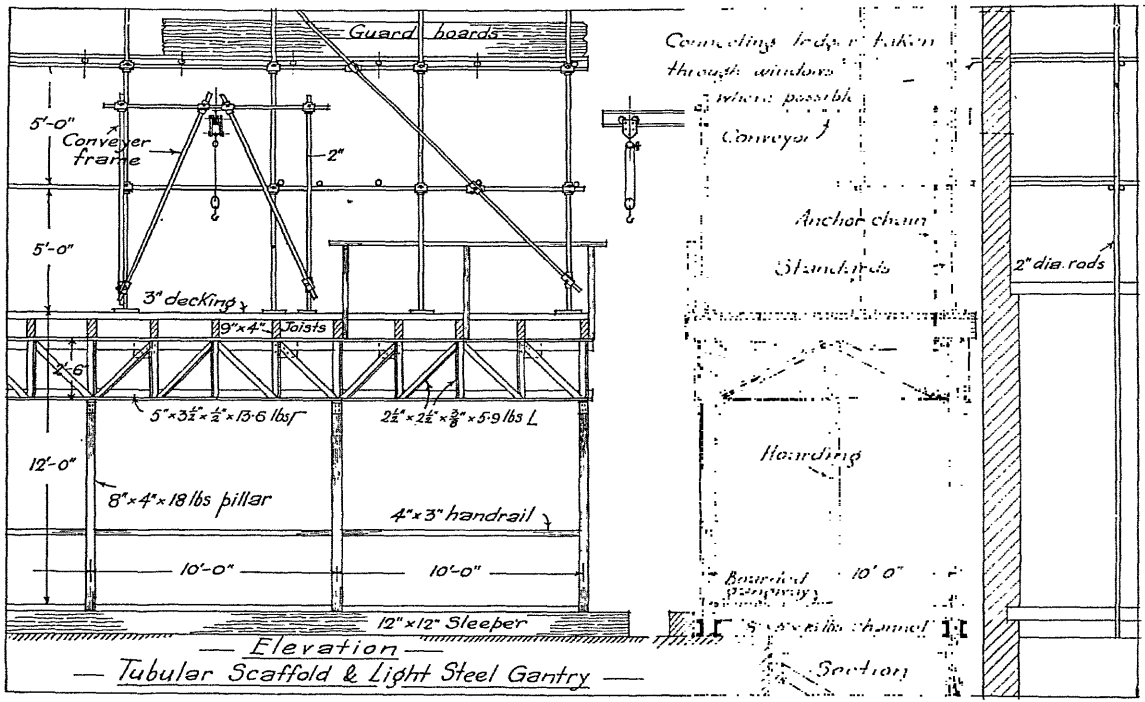


Fig. 304. Fig. 305.

Figs. 304—305.

ends ; the frames must, therefore, be made to stand independently of each other (as shown in Fig. 306) ; these are usually made of balk timber. The members are fitted together accurately, every care being taken to make the frames rigid. Iron rails are bolted to the heads of the frames upon which the travellers move ; these are turned up at the end of the gantry, to prevent the traveller moving beyond that point.

The travellers consist of two trussed beams, the arrange-

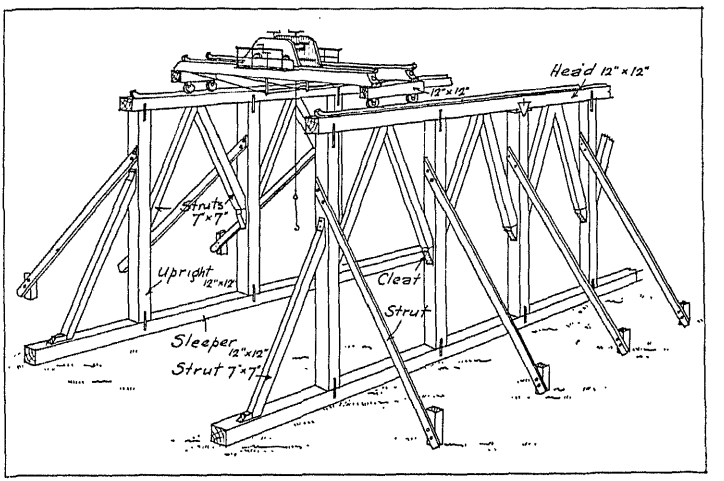


Fig. 306.

ment of the truss varying with the span ; these are connected at their extremities by short pieces of balk timber mounted upon wheels. Rails are bolted to each of the beams on which a crab, mounted on wheels, is free to move the whole length of the trussed beams.

Motion is imparted by manual, steam or electric power to the traveller by means of a cog-wheel keyed on the inside of the bearings to the axle of one of the wheels upon which the traveller moves at each end of the trussed beam ; these cogs are geared to another wheel keyed to a shaft of about 1½ inches square. Upon the square shaft a bevel

wheel is fitted, so that it is free to slide along the bar, the wheel being connected to the movable crab by means of a projecting bracket; this cog is geared to another bevel wheel keyed to an upright shaft, which is caused to revolve by the man in charge of the winch. In consequence of this apparatus requiring to slide along the whole length of the shaft the latter has to be free from end to end; if the shaft be long there would be a danger of it sagging; to obviate this it is supported by special bearings consisting of a bar, which can rotate on a pin. The lower end of the bar is provided with a weight which normally keeps the bar in an upright position. When the winch in its travel arrives at a bearing it pushes it to one side. When it has passed the point, the weight at the lower end of the bearing bar causes it to rotate back into position again.

Gantries of this description are now largely made of steel instead of timber. The uprights are made from rolled steel beams of a resistance to meet the demands connected together with a rolled steel beam at the top on which the rails are mounted. The lower ends of the pillars or uprights have suitable steel bases and are embedded in concrete. The connections between the main beams and the pillars are made rigid, and the frames are connected at their extreme ends by cross joists. This enables much of the strutting and bracing to be eliminated (see Fig. 306). For large gantries the winch is usually steam or electrically operated.

Derrick Cranes.—On tall buildings and buildings covering large areas, especially in urban districts, the problem of hoisting and transferring material from the street to its position on the site is one that considerably affects the cost of the work and the rapidity with which it can be carried out. The derrick in one or other of its forms has proved to be the most efficient appliance for the purpose. There are three types in common use, known as (1) the Scotch derrick, (2) the tower derrick, and (3) the guy derrick, either of which forms may be operated by hand or power.

Derrick Stagings.—Derrick cranes are now largely used owing to the facilities they offer for transferring materials

to any part of a building in process of erection, and to the great area commanded by them. They are elevated to the required height on three timber or steel supports, or towers, the latter being constructed in one of two different ways.

Derrick Crane.—The derrick, as shown in Fig. 307, consists of four parts: (1) The mast, (2) the jib, (3) the sleepers, and (4) the stays.

(1) *Mast.*—The mast is the upright member; this may be a braced steel member or out of one piece of timber, or out of two pieces strutted apart and braced. This terminates in a pivot on top and bottom, which allows it to rotate freely. A short distance from the bottom is fixed the machinery for raising the materials; this may be worked by hand or power—in the latter case the engine or motor is attached to a platform, to which the bottom of the mast is fixed and rotates with the mast, and beneath this platform is the gearing to change the direction of the derrick in a horizontal plane.

(2) *The Jib.*—The jib is the member to which the weights are hung; it may be a braced steel member formed of one piece of timber, or built up in a similar manner to the mast; it is attached to the mast at its lower end by a hinged joint, which allows it to work up and down, or change its direction in a vertical plane; at its other end is a wheel, over which the rope or chain that supports the weights is passed, and to this part also the rope or chain is fixed that raises or lowers the jib. Both these chains pass over pulleys at the upper end of the mast, and from there down on to the drums on to which they are wound.

(3) *Sleepers.*—There are two sleepers, consisting of timbers which lie on the ground or staging; these are connected to the lower end of the mast by a swivel joint.

(4) *Stays.*—The stays are similar timbers to the above, connected at their upper ends to the top of the mast by a swivel joint, and at their lower end to the free end of the sleepers by a link joint, thus forming two triangular frames.

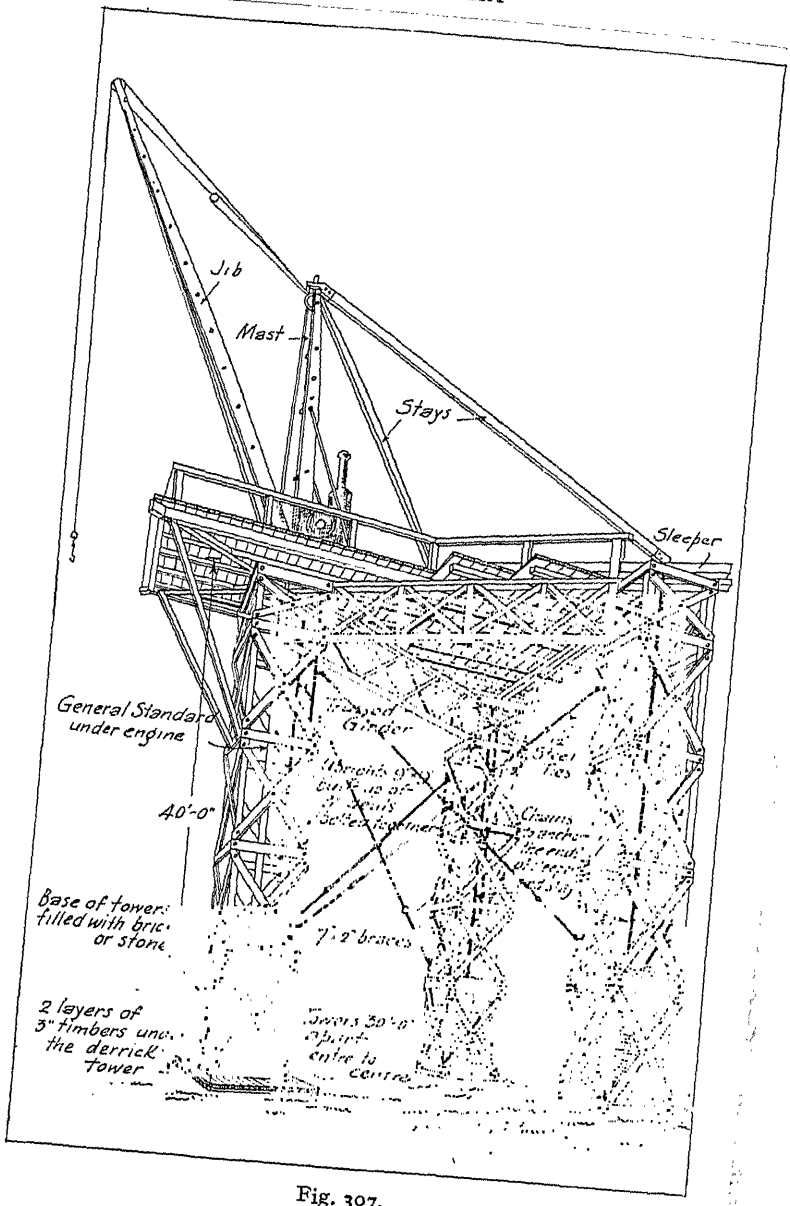

Jib
Mast
Stays
Sleeper
General Standard under engine
40'-0"
Base of tower filled with brick or stone
2 layers of 3" timbers under the derrick tower
7" x 2" braces

Fig. 307.

The stays and sleepers at their junction are anchored down to the ground, or staging, to counteract the overturning tendency of the mast when the derrick is loaded.

Derrick Towers.—These, as previously stated, are built in two methods—(1) by building three timber towers, connected by trussed beams, which support the platform or stage; one of the towers is so arranged that it has the pivot of the spinning gear centrally above, the other two are arranged so that with the first they form an isosceles triangle in plan, the junctions of the sleepers and the stays being placed respectively centrally above a tower; (2) by upright timbers properly braced and strutted in lieu of the towers above mentioned, and similarly disposed as regards the remainder of the staging.

(1) Each of the towers is constructed about 6 feet square in plan, and they are placed about the apices of an isosceles triangle, the distance between them varying with the available space and the length of the timber sleepers. Each tower consists of four uprights about 9 inches square, either out of solid timber or built of 9″ × 3″ deals bolted together, the latter being the better plan when the posts are required of a great length; these are connected together by cross-pieces or transoms, out of about 9″ × 3″, placed about 7 feet apart, dividing the towers into a number of bays, these being stiffened by cross braces, out of about 7″ × 2″ cut between the transoms and bolted to the uprights.

The towers are placed on a wooden platform laid on the ground to serve as a foundation.

The tower supporting the mast is connected to the other two by trussed beams, constructed as follows: Two booms of balk timber are placed, the upper ones on the top of the staging being halved at their intersection on the above-mentioned tower, and projecting about 4 feet beyond on either side; the lower booms take their bearing on the first transom from the top. The two booms are connected by iron bolts placed at distances of about 5 feet apart, with cross braces cut between; the remaining side is connected by a single balk, with two inclined struts if the span be large.

The derrick is anchored by means of chains, which are passed over the sleepers at the top of the staging, and are connected to the platform at the bottom of each tower, which is loaded with bricks, stones, or other material to a weight of at least twice the load to be raised ; the chains may be tightened by means of coupling screws.

To prevent the towers and platform racking when the crane is loaded, the towers may be braced on two sides as shown in Fig. 307, or better still on the three sides either by lashing scaffold poles or bolting square timbers to form braces, or preferably by steel ties. If the towers are very tall, say, above 70 feet, horizontal struts about the towers should be fixed at the level where the braces intersect.

(2) In the second method, commonly applied in Scotland, a balk of timber is used in the place of each built-up tower, these having raking shores on all their sides ; the arrangement for supporting the platform is similar to that described above, taking care to fasten the sleepers direct on to the uprights.

In the latest type of Scotch derrick, the three towers, as well as the parts of the derrick, are constructed of steel (see Fig. 308).

The tower derrick differs from the Scotch in having only one tower, a great advantage where the three towers of the former type would interfere with the internal arrangements of building during construction. In this case the mast, engine, sleeper, jib and backstay are situated in one vertical plane ; they are fixed on one base, and all rotate in a horizontal plane together. At the extreme end of the backstay and sleeper is the counterweight proportioned to balance the load and the jib when in its most extended position. The tower is usually stayed with guys to increase its stability during lifting or derricking operations (see Fig. 310).

The guy derrick consists of a mast mounted with the hoisting tackle and motor on a rotating platform. The mast is stayed in a vertical position with four steel guys. The latter divide up the area about the mast into four sections.

The jib is attached to the lower extremity of the mast. The jib can operate in either of the four sections, but

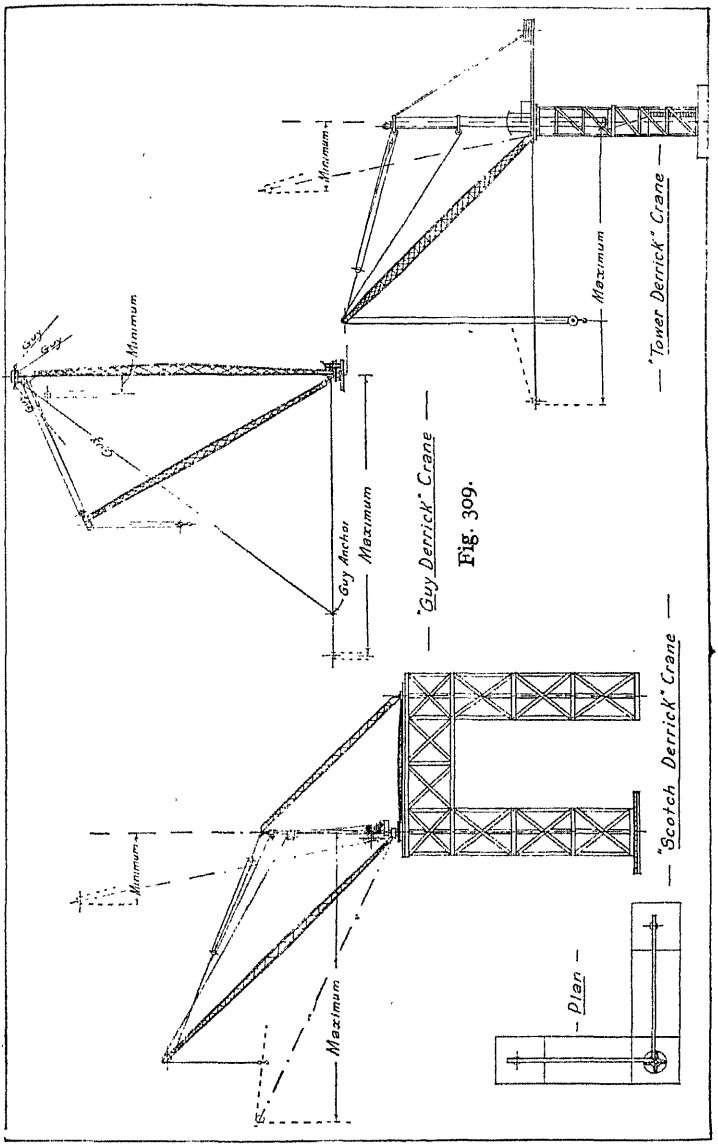


Fig. 310.

Fig. 309.

Fig. 308.

Figs. 308—310.

only in one at a time. To pass to another section, it must first be raised to a vertical position. Like the tower derrick, it can command the whole area of operations (see Fig. 309).

Cranes and hoisting tackle have now become a specialist business, and for further information and details see the British Standard Specification No. 327 Parts I. and II.

Lifts.—In buildings where it is impracticable to employ derricks it is usual to construct lifts. The shafts and loading platforms are usually projected from the ordinary scaffold staging if on the exterior of the building, or up staircase wells in the interior. They are generally operated by power plant from the base. The lifts usually operate in pairs. The bricks, mortar or other materials are loaded in barrows and sent up one lift and an empty is sent down the other. On reaching its destination the material is distributed to the various parts of the building on runways constructed for the purpose. The tubular scaffolding is especially adapted for this purpose.

Formwork.—Since steel frame and ferro-concrete construction have been introduced the preparation of the formwork or moulds has become an important part of temporary carpentry work, a part demanding considerable skill and ingenuity both on the part of the designer and of the carpenter in the preparation and erection of the several units.

The timber should be good sound air-seasoned stuff, preferably prepared on the four sides, sufficiently thick to resist the pressures exerted by the concrete when first poured without bulging or deflection. While the parts must be accurately made, it should be borne in mind that it is not cabinet work, and when nailing the parts together it should be remembered that it has to come apart again with the least possible injury to the material, so that it can be used several times over.

There is a close resemblance in the formwork in both steel frame and reinforced concrete construction. The essential difference is that with the steel frame, the centering for the floors is suspended from the beams themselves. This is done so that the beams may take up whatever deflection is due to the dead load while the concrete is in

a plastic condition. In the ferro floors this is not possible, so that the formwork must be shored up from the ground upwards. The forms for the several units in both steel and ferro work are similar, and although every job has its peculiarities, the same principles of construction are observed throughout. Taking the case illustrated in the chapter on ferro work, which has pillars, main beams, secondary beams and slabs, the construction of the units will be given first, and finally the parts when assembled.

Column Forms (see Figs. 311, 312).—Column forms consist in the case of rectangular columns of four shutters nominally out of 1¼-inch stuff, 1⅛-inch finish; the thickness must be proportioned to the height of the column as there is a considerable bursting stress at the bottom of the forms until the concrete has set. The exact hydrostatic pressure of the semi-liquid concrete is indeterminate; the pressure varies according to the sloppiness of the mixture, but the force exerted is probably in the region of an equivalent liquid of from 100 to 120 lbs. per cubic foot; the actual stress at any point in the height of the column is equal to the w × h where " w " is the equivalent weight of the mixture and " h " is the height from the point to the top of the column.

The sides of the column casing are held together by a series of yokes, see Fig. 311, the number and spacing of which depend upon the thickness of the column casing. There are two things to determine: (1) The greatest length of the column casing to safely resist the stress: this will give the spacing of the yokes. (2) To determine the dimensions of the yokes.

Then, taking the height of the column to the bottom of the deepest beam connected to it is the height to which concreting would normally cease, till the beam and the next length of column reinforcements were in position; by this time the setting action would be complete and the stress on the column casing would be relieved. Let the height be 10 feet and the equivalent weight of the semi-fluid concrete be taken as 130 lbs. If the first yoke is placed 4 inches above the bottom then the stress " p " would be

$$p = wh = 130 \times 9{\cdot}66' = 1250 \text{ lbs. ft.}$$

Let the stress be considered uniform between the yokes, an error which would be on the right side. Consider the casing as a continuous beam. Then $M_l - M_R$

$$\frac{Wl}{10} = \frac{(1250 \times l) \times l}{10} = \frac{fbd^2}{6}$$

Take $f = 1200$ lbs./sq. in. and $b = 1''$.

$$125\, l^2 = \frac{(1200 \times 12^2) \times 1 \times \frac{1 \cdot 125^3}{12^2}}{6}$$

$$l = \sqrt{2 \cdot 03'}$$
$$= 17 \cdot 15''$$
say $18''$

Then if "p" at the next depth $120'' - (18 + 4) = 98''$ be worked in a similar manner the yoke spacings will be

$$4''\quad 18''\quad 18''\quad 22''\quad 24''\quad 32''$$

Thus six yokes would be required up to 10 feet in height.

There are many devices for connecting the sides of the column casing, but that shown on Fig. 311 is probably the simplest. The distance between the bolt holes, which represents the span, can be taken as 12 inches more than the diameter of the pillar.

In this case, if the pillar be taken as 16 inches length of side, then the span of the yoke would be 28 inches, the central 16 inches of which would be under a distributed load of 1,250 lbs. per foot run $= 1{,}250 \times 1 \cdot 33 = 1{,}660$ lbs.

The reactions R on the bolts = 830 lbs.

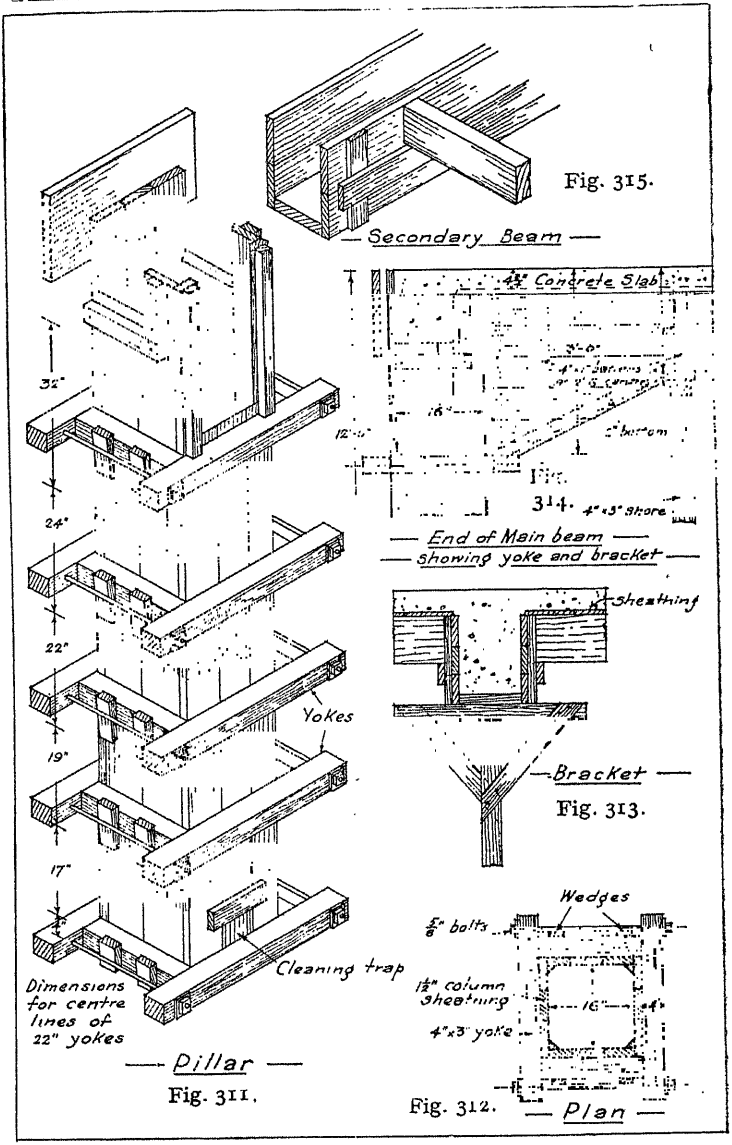
$$w = \frac{1660}{16} = 104$$

Then $$M = (830 \times 14) - \frac{104 \times 64}{2}$$
$$= 8272 \text{ lbs. ins.}$$

Taking the breadth of the yoke as $3''$.

Then $$8372 = \frac{1200 \times b \times d^2}{6}$$

$$d = \sqrt{\frac{8372 \times 6}{1200 \times 3}}$$
$$= 3 \cdot 8'', \text{ say } 4''.$$


Fig. 315.
Secondary Beam
4½" Concrete Slab
32"
24"
22"
19"
17"
Yokes
Cleaning trap
Dimensions for centre lines of 22" yokes
Pillar
Fig. 311.
Fig. 314.
4"×3" Shore
End of Main beam
showing yoke and bracket
Sheathing
Bracket
Fig. 313.
Wedges
⅝" bolts
1½" column sheathing
16"
4"
4"×3" yoke
Fig. 312.
Plan

Figs. 311—315.

Use $3'' \times 4''$ for the yokes throughout. Use $\frac{5}{8}$-inch bolts with an effective area of say $\frac{1}{8}$ inch. It is unnecessary to calculate these as the strength is ample, and it is not advisable to use any bolts under $\frac{1}{2}$ inch diameter. See Fig. 312.

Main beams.—The internal dimensions of the beam casing is that of the beam less the thickness of the slab. The bottom is made the width of the beam and about 2 inches in thickness; if the width is wider than the market dimensions the material is battened together; chamfered feathers are nailed on the edges. The sides are formed of stuff similar to the pillar casing, say $1\frac{1}{8}$ inch finish; if tongued and grooved a better job will result. The beam is taken in this case as 20 inches deep, 12 inches breadth, with a haunch 36 inches deep. The slab is $4\frac{3}{4}$ inches thick. The sides are nailed on to the edge of the bottom. The width therefore would be $20'' - 4\frac{3}{4}'' + 2'' = 17\frac{1}{4}''$. This would best be made up from three boards to distribute the shrinkage and expansion more equably. The boards are battened with say $4\frac{1}{2}'' \times 1''$ battens at about 3 feet intervals. The construction of the haunch is shown in Fig. 314. In erecting, the bottoms are first placed in position between the pillar casings; if there is a haunch, this rests on the top yoke of the pillar; the bottom is then supported by the crossheads of the posts, to which it is tacked, being carefully levelled by wedging from the lower end of post: it is usual to give the bottom a slight camber of about $\frac{1}{4}$ inch to $\frac{3}{8}$ inch in 10 feet, to take up any settlement. The sides are then placed in position, being nailed to the sides of the bottom planks, and secured to the pillar casings as shown in Fig. 313. The sides are braced apart by temporary strainers to their correct distances.

The apertures for the secondary beams are now cut in the sides to the correct dimensions of the beam sections and the bearer nailed on to receive the bottom plank of the beam; this is placed in position and supported on the crossheads of the posts. The sides are nailed on as in the case of the main beams. A ledger about $4'' \times 1\frac{1}{4}''$ is nailed to the battens of the side casing to form a bearing for the slab joists. The slab joists are placed from 2 ft. 6 in. to

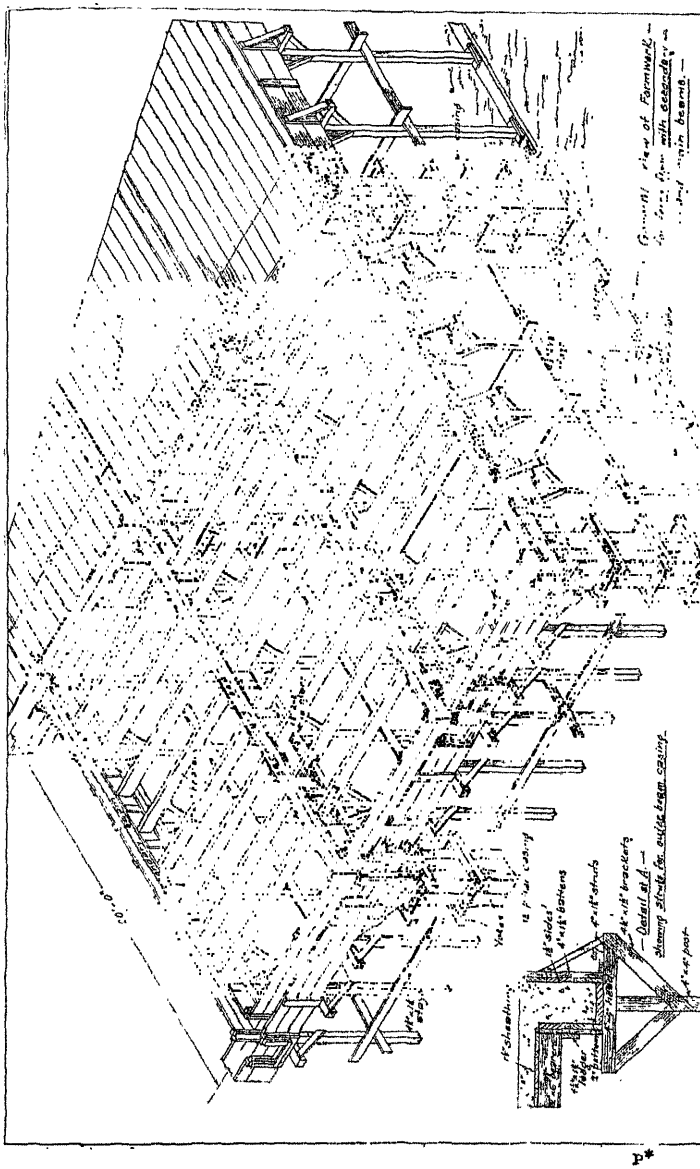


Fig. 316.

Fig. 317.

Figs. 316—317

3 feet apart on the ledgers. Their dimensions vary with the weight they have to carry. Taking 1 inch boarding for the slabs with a load of 80 lbs. per foot super, which allows for the weight of the concrete and any incidentals during the depositing, the bearers would require to be 2 ft. 9 in. apart, say 2 ft. 6 in. The load on these bearers in this case taken as 9 feet in length would be $9' \times 2{\cdot}5 \times 80 = 1{,}800$ lbs.

Then $$\frac{Wl}{10} = \frac{fbd^2}{6}$$

let the breadth $= 2''$

$$\frac{1800 \times 81}{8} = \frac{1200 \times 2 \times d^2}{6}$$

$$d = \sqrt{\frac{1800 \times 81 \times 6}{8 \times 1200 \times 2}}$$

$$d = 6{\cdot}75'', \text{ say } 7''$$

Use 8 joists $2'' \times 7''$ at $2'\ 6''$ centres.

Fig. 317 shows the parts assembled. The formwork for ferro work must be accurately plumbed and levelled and firmly braced to prevent any movement, and the whole should be checked before the pouring in of the concrete begins.

Forms for Steelwork.—The example taken is a floor with main girders consisting of $18'' \times 6''$ R.S.J.'s at 12 feet centres and a ferro-slab 6 inches thick, with 2 inches of concrete above the girder. Let the boarding for the slab be 1 inch with bearers at intervals of 2 ft. 6 in. Take the load as 80 lbs. per foot super. Then the load on each joist will be $W = 11 \times 2{\cdot}5 \times 80 = 2{,}200$ lbs.

Then $$\frac{Wl}{8} = \frac{fbd^2}{6} \quad \text{let } b = 2''$$

$$\frac{2200 \times 132}{8} = \frac{1200 \times 2 \times d^2}{6}$$

$$d = \sqrt{\frac{2200 \times 132 \times 6}{8 \times 1200 \times 2}}$$

$$= 9''.$$

Use $9'' \times 2''$ joists, 2 ft. 6 in. centres.

The sides will be battened with say $4'' \times 1\frac{1}{4}''$ at 30-inch

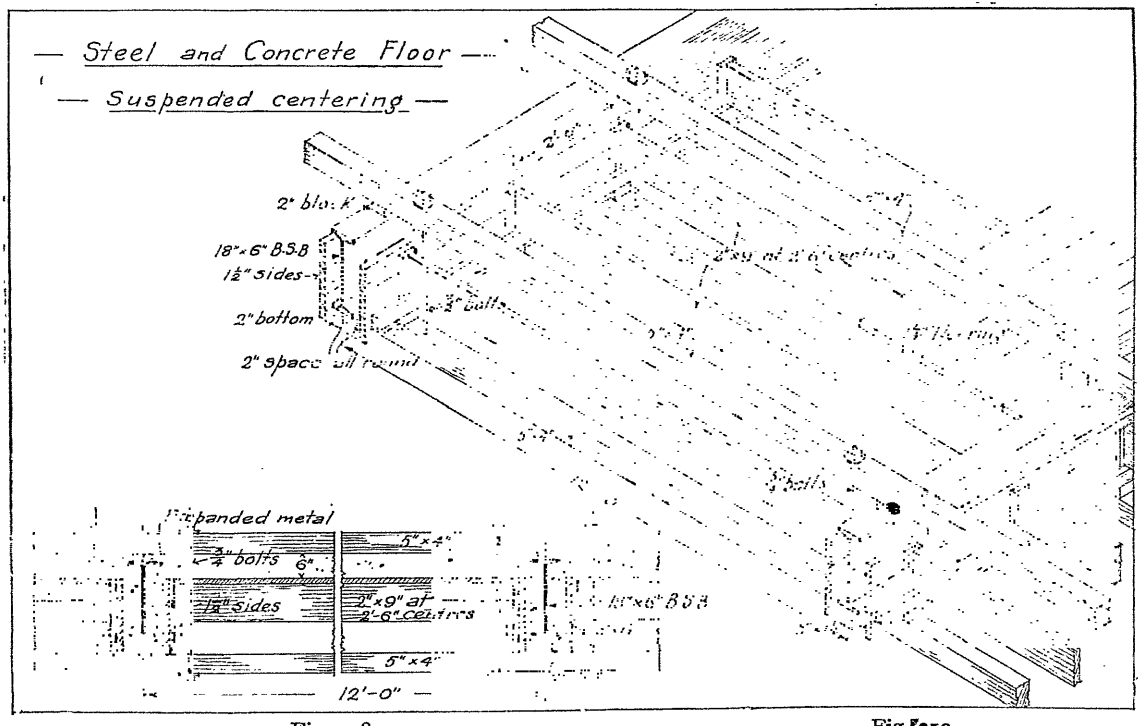


Fig. 318.

Fig. 319.

Figs. 318—319.

intervals; at 9 inches down the batten fix bearers out of say $4\frac{1}{2}'' \times 1\frac{1}{2}''$; on these place the joists to receive the slab boarding. See Figs. 318 and 319.

The whole of the casing and boarding will be supported by suspended quarterings placed at say 5 feet intervals. Use $\frac{3}{4}$-inch bolts and $4'' \times 5''$ quarterings. The upper quartering to be blocked about 2 inches above the top of the girder.

CENTERING

Definition.—A framework of timber arranged to support temporarily the voussoirs of arches during their construction is termed a centre; they must be constructed sufficiently strong to resist any deforming stresses, and require to be carefully and rigidly put together; but as they are only for temporary work, care should be taken to frame them so as to do the timber the least amount of injury. Centres consist of ribs supporting a curved surface made to the outline of the soffit of the intended arch, together with braces and ties arranged to prevent the least appreciable deformation.

Classification.—Centres are classified under two heads: (*a*) those with built-up ribs; (*b*) those with solid ribs. The first are used for spans up to about 20 feet, the second for spans above that dimension.

Ribs.—There are two methods of building the ribs: (*a*) By short pieces of boarding about 1 inch thick cut to the required curve—the joints are made normal to the curve; the rib is made in two thicknesses nailed together, with the pieces overlapping, as shown in Fig. 320; (*b*) ribs are made in one thickness in the larger centres out of timber from about 3 inches thick and upwards, the pieces being connected together by dogs or preferably by iron plates screwed over the joint, as shown in Figs. 322 and 325. This centre is designed to be supported at two intermediate points in addition to the extremities to prevent deformation during the building.

Ties.—The ribs are secured at their extremities, to

prevent them spreading, by pieces of timber spiked or bolted to them, as shown in Figs. 320 and 324.

Braces.—These are required to support the ribs at intervals to prevent any deformation in the curve; they must be capable of withstanding alternately compressile and tensile stresses. Let a centre be complete and in position, and the arch commenced as is usual at both sides, these being carried up simultaneously till they meet in the centre. There is practically no stress transmitted to the centre till the angle of the bed joints exceeds the angle of repose of the material of which the voussoirs are composed, but when this is passed there is a compressive stress exerted at the haunches tending to make that part sink and the crown to rise; this stress is so great in the large centres that it becomes necessary to weight the crown while the haunches are being built. When the arch is nearly completed, that is, just before the keystone is inserted, the stresses are reversed, a compressive stress being exerted on the braces at the crown and a tensional stress on those at the haunches. Figs. 320 to 324 show the usual methods of securing the ends of the braces.

Laggings.—Strips of wood are nailed to the ribs to form the surface to support the voussoirs called lagging pieces; these vary in size according to the weight to be carried and the distance the ribs are placed apart, from $1\frac{1}{2}'' \times 1''$ in the smaller centres to about $4'' \times 4''$ in the larger types.

The laggings for rough brick or stone arches are usually placed a short distance apart, but for gauged brick arches they are fitted and placed close together, the joints being cleaned off and the curve made true with the plane, in order that the position of each voussoir may be marked thereon. The surface may also be formed by two layers of $11'' \times \frac{3}{8}''$ pine, bent about the curve and fixed to the rib, as shown in Fig. 320. 3-ply is now frequently used.

Centres for stone arches where the voussoirs are large have no laggings, but the ribs directly support the voussoirs; they are gauged for their correct position by means of a radius rod, and are packed up by means of wedges, as shown in Fig. 320.

Figs. 322 to 326 show a centre and details suitable for spans from 25 feet to 40 feet. In this the rib is built up of solid members, as shown in the details. This centre is an economical construction where the space beneath the arch is not required during construction. Figures 327 to 330 show a form of centre much used where the space beneath the arch is required for traffic during its construction. This centre is supported at the sides only, and the main tie has been raised considerably in order to give greater headroom. Such centres as these are suitable for the construction of bridges; the ribs are usually placed about 4 feet apart and the laggings are out of stuff about 5″ × 3″, and a pair of wedges is placed under the feet of each rib.

Supporting and Easing of Centres.—Provision must be made for the gradual easing of centres, in order to allow the arch to take its bearing gradually; this may be accomplished by means of lifting jacks or wedges, the latter usually being adopted; the arrangement is as follows: The ribs are fixed to a plate, extending the length of the centre; these rest upon pairs of folding oak wedges, which in their turn are supported by similar horizontal plates supported by uprights, which rest with their lower end on the ground or other solid support. The folding wedges between the horizontal plates are greased if the centres be heavy, and are for the purpose of easing the centre gradually; these are shown in Fig. 326.

Steel Centres.—For very large arches the ribs of the centres are now frequently constructed of rolled steel joists bent to the required curve, and trussed beneath to meet the requirements of the case. They are supported at their extremities on screw jacks for easing on completion.

Centres for Intersecting Arches.—The centres for barrel headed vaults are arranged at their intersection with other vaults as follows: The centre for the main vault is made in the usual manner, the distance between the ribs varying with the weight to be carried, the whole of the ribs being covered with the laggings. The centre extends beyond the line of intersection of the side vaults at each side, and is

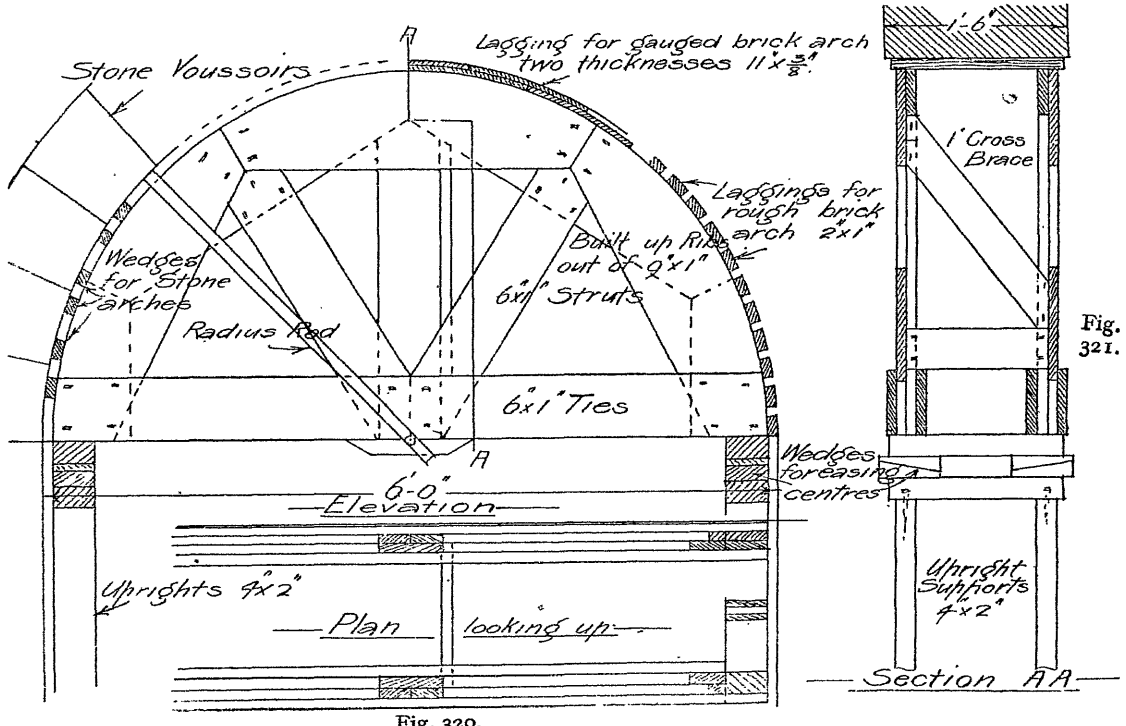


Fig. 320.

Fig. 321.

Figs. 320—321.

Fig. 322.

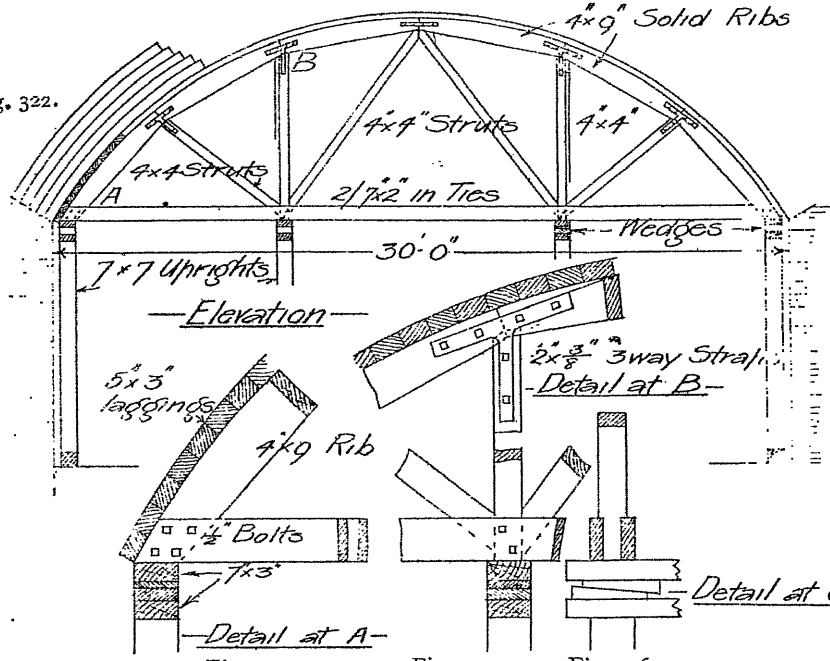
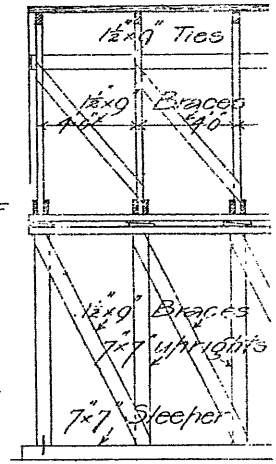


Fig. 324. Fig. 325. Fig. 326.



Section through Centre

Fig. 323.

Figs. 322—326.

fixed into position. The centre for the side vault is now constructed, one rib being placed in contact with the main centre at the springing points, the others at the requisite distance apart; the laggings are then laid on, extending in each case past the end rib till they touch the surface of the main centre, to which they are scribed and fixed. Should the distance between the end rib and the surface of the main centre exceed the distance between the ribs, a backing piece is fixed to the main centre to support the laggings.

Groined Vaulting.—The centres for groined vaulting are made in two ways, depending upon the construction of the vault. If the bed joints and courses of the vault be laid horizontally the centres are constructed as above described, provision being made at the intersections for the groin-stones projecting below the surface of the vaults; but as in the case with many pointed intersecting arches, where the panels form part of a domical or an ellipsoidal surface, wood ribs to support the groins only are made; the stone ribs are built first, and are rebated along their upper edges to receive the stone panels. As each course in the panels is similar to a portion of a bed course of a dome, it only requires supporting when the bed joints exceed the angle of repose of the material, and then only till each course is complete, or takes its abutments, as in this case, against the groined ribs. The support for this work usually consists of a curved wood rule held in position.

Where the vaults are constructed of ferro-concrete the whole surface of the centre must be covered with wood sheathing.

SHORING

Definition.—Shoring is the art of temporarily supporting structures that are in an unsafe condition till such time as they have been made stable, or in supporting walls, the lower part of which has been removed to allow of a large opening to be made and which is to be spanned by an arch or girder, till the construction or fixing of the latter has been completed.

Fig. 327

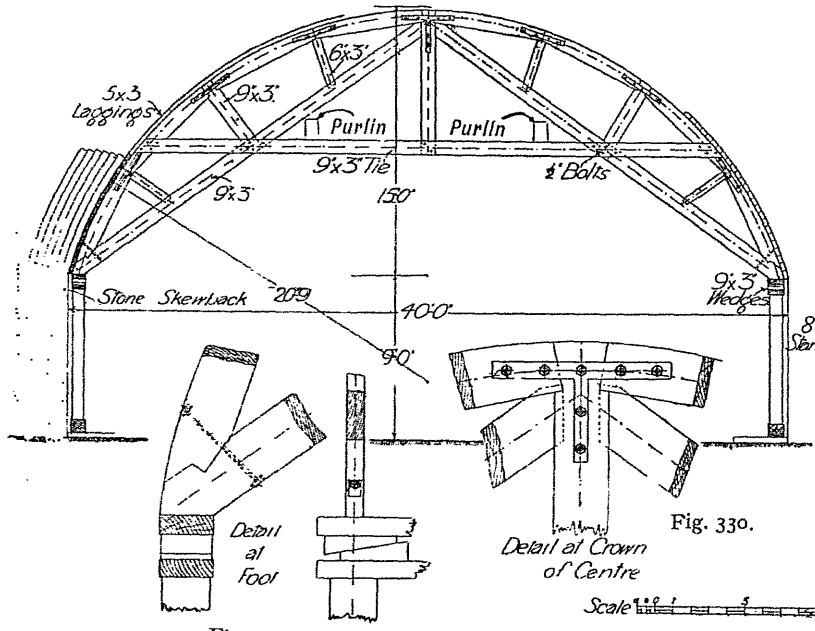


Fig. 328.

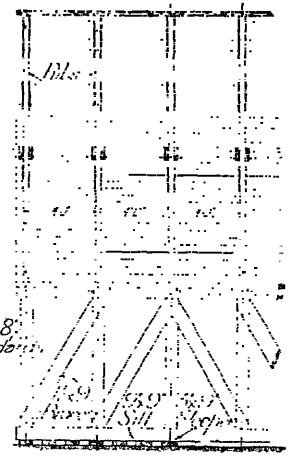


Fig. 329.

Fig. 330.

Figs. 327—330.

Object of Shoring.—The object of shoring is to prevent dangerous walls developing and continuing symptoms of failure, and to retain the unstable position till they can be more permanently secured. Shores would only be used in exceptional instances to straighten walls.

Theory of Shoring.—Walls when shored may be considered to be acted upon by the following forces: viz., vertical, horizontal, and inclined, thrusts of floors, roofs, shores, or other forces; and to maintain equilibrium it is necessary to know the greatest disturbing force that can be exerted by the thrust of roofs, floors, etc.; in order that the necessary shores to equilibrate and counteract the disturbing force may be determined, no intended equilibrating force must be greater than the disturbing force or it will have the tendency to act detrimentally to the stability of the wall. The shores required may be determined by the principle of the moments.

In practice it is impossible to determine the amount of the actual disturbing thrust tending to overturn the wall, but the maximum value of the overturning thrust capable of being resisted by the wall can be determined and provided for.

Classification.—There are three general systems of shoring, known as—(1) raking, (2) horizontal or flying shores, (3) dead or vertical shores.

(1) *Raking Shores.*—These consist of pieces of timber placed in an inclined position with one end resting against the faces of defective walls, the other upon the ground, the most convenient and best angle for practical purposes being 60 degrees, from which they vary to 75 degrees; the angle is often determined in urban districts by the width of the footway.

These shores are fixed in systems of one or more timbers placed in the same vertical plane inclined at different angles, and supporting the building at varying levels.

The horizontal distance between the systems in dead or unperforated walls is usually not more than 8 feet; but on walls pierced with windows they are placed on the intervening piers.

A wall-plate, consisting of a 9″ × 2″ or 9″ × 3″ deal, is placed on the wall to receive the ends of the shores being fixed to the wall by means of wall hooks driven in the joints of the brickwork. The wall-plates should be in one piece throughout the system ; if owing to their length it is necessary to have them in two pieces they should be halved and securely spiked as shown in Fig. 334. To form an abutment for the end of the shores, needles, consisting of pieces of 4″ × 3″, cut as shown in Fig. 331, are passed through a mortice made in the wall-plate, and projecting in the wall at least 4½ inches, a half-brick being taken out to receive them. The following considerations determine the position of the needles : The end of a raking shore should only be placed where there is something such as a floor or roof at the back of wall to resist the thrust, otherwise the walls are liable to bulge inwards at that part ; there is also a danger near the top of the wall of that part being pushed off if it be not sufficiently heavy. The centre line of shores should, therefore, be made to point directly below the wall-plates if they should be on the wall in question ; but if the joists should be parallel to the wall, the centre lines of the floor, wall, and shore should meet in a point. The needle should be placed so that the pressure exerted by it takes place along the centre line of the shore. The shore should be notched out at its upper end to receive the needle, thus obviating any tendency to lateral motion. The needle is also further supported at its top side by a cleat nailed on the wall-plate, as shown in Fig. 332.

Sole Piece.—The feet of the shore rest upon a sole plate usually embedded in the ground in an inclined position, and consists of a piece of 11″ × 3″. The inclination of the sole plate must not be at right angles to the shore, the rule in practice being to fix it 1 in 24 out of the perpendicular to the shore to enable the latter to be tightened up gradually by means of a crowbar. On soft ground the sole plate is bedded on a platform of timber to distribute the pressure over a greater area. The shores are tightened up by means of a crowbar inserted in a slot made in the foot of the shore, as shown in Fig. 333 ; wedging should not be resorted to here, as the vibration caused would be

Fig. 332.

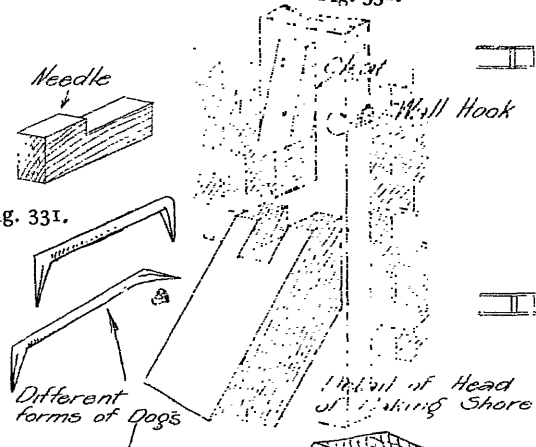


Fig. 331.

Fig. 333.

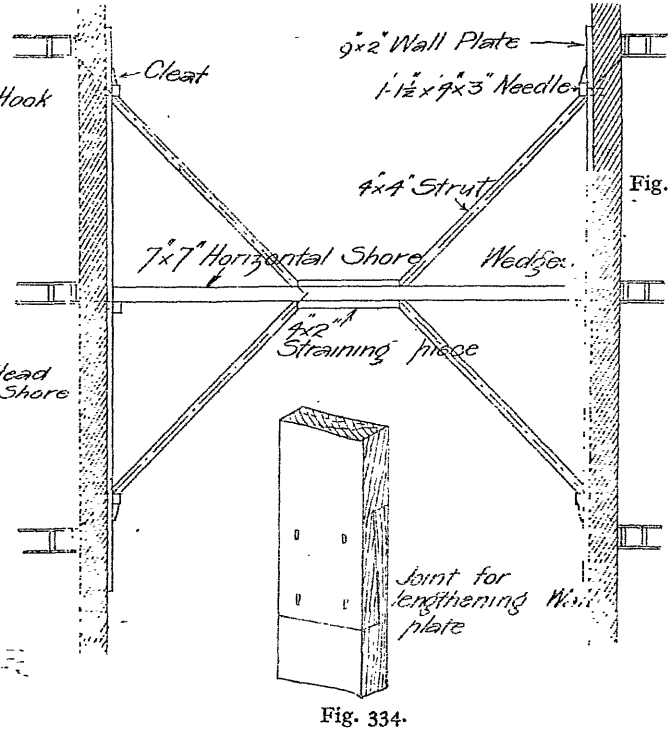


Fig. 33

Fig. 334.

Figs. 331—336.

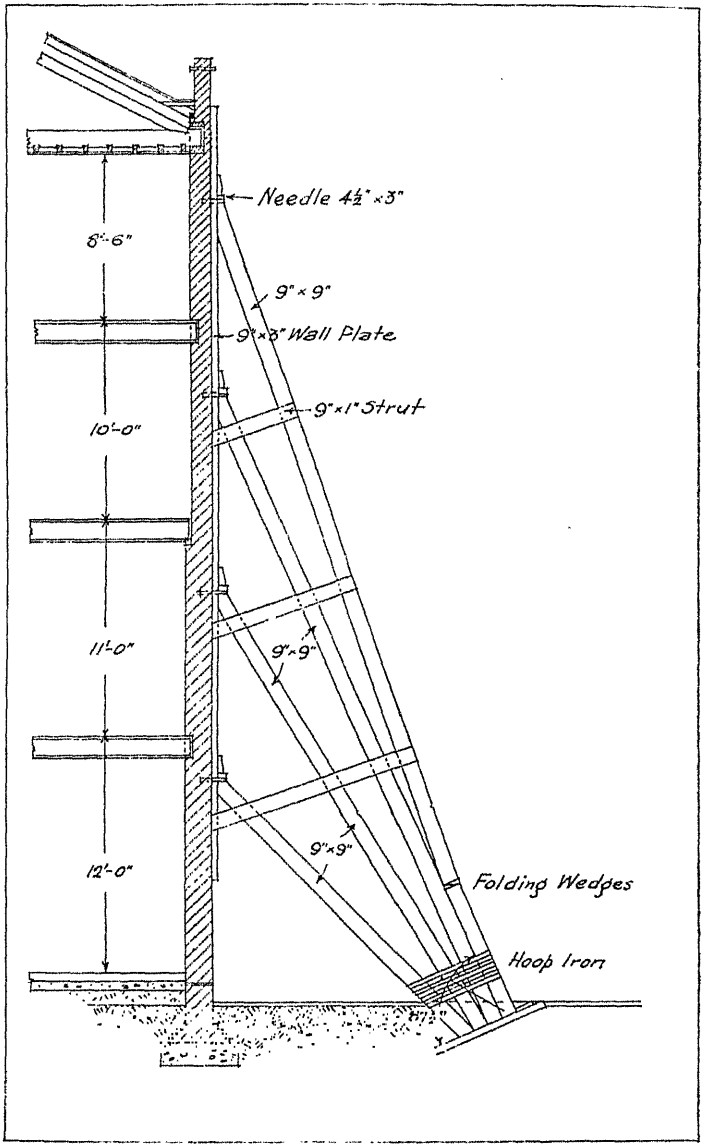

Needle 4½" × 3"
8'-6"
9" × 9"
9" × 3" Wall Plate
9" × 1" Strut
10'-0"
11'-0"
9" × 9"
12'-0"
9" × 9"
Folding Wedges
Hoop Iron

Fig. 337.

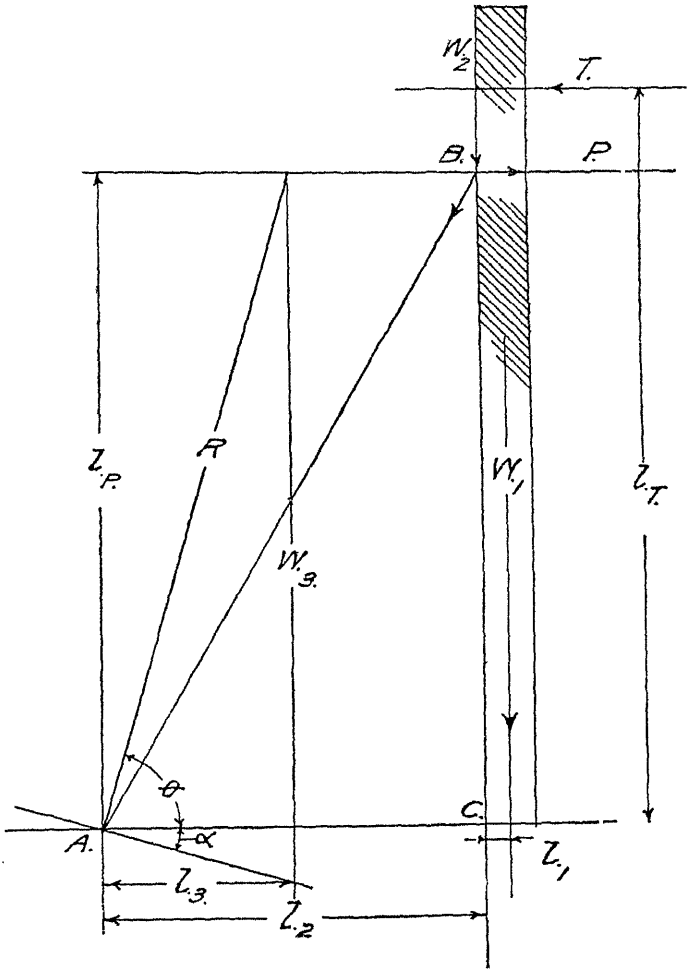


Fig. 338.

detrimental to the already unstable building; the shore should only be forced tight, but not enough to disturb the wall.

When the shore is in position it is secured to the sole piece by an iron dog, and a cleat is nailed on the sole piece in front of the shore, Where more than one shore is used in a system, the bottom ends are bound together either by hoop iron or pieces of boarding nailed across the whole of them on each side to connect them all at this part. At intervals in the height, boards are nailed to the sides of the shores and the wall-plate—these are called struts; they have the effect of binding the whole of the pieces together, and of stiffening the shores considerably, as shown in Fig. 337.

Theory of the Raking Shore.—The actual thrust on a raking shore is indeterminate, but the maximum thrust that the wall can sustain so that it is just on the point of overturning can be determined, and if this be taken and from this the thrust on the shore be found, the error will be on the safe side, for if a greater thrust than this were to exist the wall would already have failed.

Referring to Fig. 338.

T = Overturning thrust on wall.
P = Thrust of shore normal to wall.
W_1 = Weight of wall.
W_2 = Weight above needle.
W_3 = Weight of shore.
R = Resultant thrust on sole plate.
l_T = Leverage of overturning thrust.
l_P = Leverage of thrust of shore.
l_1 = Leverage of wall $= \frac{t}{2}$
l_2 = Leverage of W_2
l_3 = Leverage of W_3

Then for T take moments about the point C—

$$T \times l_T = W_1 \times l_1 = \frac{W_1 \times t}{2}$$

$$T = \frac{W_1 \times t}{2 \times l_T} \quad \ldots\ldots\ldots \quad (1)$$

Then for P the moment of P about C = the moment of T about C.

$$P \times l_P = T \times l_T$$

$$P = \frac{T \times l_T}{l_P}$$

Substituting for T from (1)—

$$P = \frac{W_1 \times .t \times l_T}{2 \times l_T \times l_P}$$

$$\text{and } P = \frac{W_1 t}{2 l_P} \quad \ldots\ldots\ldots\ldots (2)$$

Weight required in the mass of wall above the needle to prevent dislodgment by the upward pressure of the shore.

Taking moments about A

$$W_2 \times l_2 = W_3 \times l_3 \times P \times l_P$$

$$W_2 = \frac{W_3 \times l_3 + P \times l_P}{l_2}$$

Substituting for P from (2)

$$W_2 = \frac{1}{l_2}\left(W_3 \times l_3 + \frac{W_1 t\, l_P}{2 l_P}\right)$$

$$= \frac{1}{l_2}\left(W_3 \times l_3 + \frac{W_1 t}{2}\right) \quad \ldots\ldots (3)$$

Compression on shore C

$$C = \sqrt{W_2^2 + P^2} \quad \ldots\ldots (4)$$

The angle that the reaction of the shore makes with the horizontal $= \theta$

$$\text{Tan } \theta = \frac{l_P}{l_3} \quad \ldots\ldots (5)$$

The angle that the sole plate should make with ground or horizontal

$$= 90 - \theta \quad \ldots\ldots (6)$$

$$\text{Stress on shore} = \frac{\text{compression on shore}}{\text{area of shore}} \quad \ldots\ldots (7)$$

EXAMPLE.—An 18-inch wall 50 feet high has an outward thrust at 5 feet from the top. A convenient place for fixing the head of a raking shore is at a height of 40 feet from the ground. A 9″ × 9″ fir shore is fixed to support the wall at an angle of 60 degrees to the ground. Weight of brickwork 112 lbs. per cubic foot. Weight of shore 35 lbs. per

cubic foot. Determine the conditions of stability and the pressure and stress on the shore. The shores to be placed at 8 feet centres.

Then

$$W_1 = 50 \times 8 \times 1{\cdot}5 \times 1 = 30 \text{ tons}$$
$$W_2 = 10 \times 8 \times 1{\cdot}5 \times 1 = 6 \text{ tons}$$
$$W_3 = \frac{40 \times 2}{\sqrt{3}} \times \frac{81}{144} \times \frac{35}{2240} = {\cdot}405 \text{ tons.}$$

Then to determine the thrust T

$$\text{From (1) } T = \frac{W_1 t}{2 l_T} = \frac{30 \times 1{\cdot}5}{2 \times 45} = {\cdot}5 \text{ ton.}$$

The thrust of shore P

$$\text{From (2) } P = \frac{T \times l_T}{l_P} = \frac{{\cdot}5 \times 45}{40} = {\cdot}5626 \text{ tons.}$$

The required weight of W_2

$$\text{From (3) } W_2 = \frac{1}{l_2}\left(W_3 \times l_3 + \frac{W_1 \times t}{2}\right)$$
$$= \frac{1}{23}\left({\cdot}405 \times 11{\cdot}5 + \frac{30 \times 1{\cdot}5}{2}\right)$$
$$= 1{\cdot}17 \text{ tons.}$$

$W_2 = 6$ tons, and is therefore safe.

The compression on shore C

$$\text{From (4)} = \sqrt{W_2^2 + P^2}$$
$$= \sqrt{36 + {\cdot}315}$$
$$= 6{\cdot}025 \text{ tons.}$$

$$\text{Stress on shore} = \frac{\text{compression on shore}}{\text{area of shore}}$$
$$= \frac{6{\cdot}025 \times 2240}{81}$$
$$= 167 \text{ lbs. square inch.}$$

Angle θ that the reaction of shore makes with the horizontal

$$\text{From (5) } \tan\theta = \frac{l_P}{l_3} = \frac{40}{11{\cdot}5} = 3{\cdot}54$$
$$\theta = 74^\circ\ 14'$$

Angle α that sole plate should make with the horizontal or ground.

$$\text{From (6)} \quad \alpha = 90 - \theta$$
$$= 19° \cdot 46'$$

(2) *Horizontal or Flying Shores.*—These are used to temporarily support two parallel walls, where one or both give signs of failure. Thirty feet between the walls is usually considered to be the maximum length. These are used mostly in urban districts, usually where one of a number of terrace houses has to be removed, to temporarily support the houses on either side; they are erected as the old house is being removed, and are taken down when the new building is of a sufficient height to dispense with them.

They consist, as shown in Fig. 336, of a timber placed horizontally, and cut tightly between the walls to be supported, the ends resting against wall-plates placed vertically on the walls. They are stiffened by inclined braces placed above and below the horizontal shore. These stiffen the shore and add two more points of support to each wall, The method of fixing being as follows :—

Two wall-plates are fixed, one on each wall in a similar manner to those described for raking shores having a needle fixed where it is desired to place the horizontal timber, care being taken to keep this as far as possible in the line of the floors of the buildings on either side. The horizontal shore is now placed in position, having a straining beam out of about $4'' \times 2''$ nailed on the upper and lower sides. This timber rests upon the needles, and if there is any space between the end of the shore and wall-plate, a pair of folding wedges is inserted and driven up tightly. In the case of the demolition of a terrace house, the wall-plates are fixed before the demolition, then the horizontal shore when the demolition process has come down to that level. The upper braces are then fixed, and lastly the lower braces. By proceeding in this manner, the party walls are supported by the shores before the old work has been removed.

(3) *Vertical or Dead Shores.*—Shores placed vertically are termed dead shores. They are used for temporarily supporting the upper parts of walls, the lower parts of which

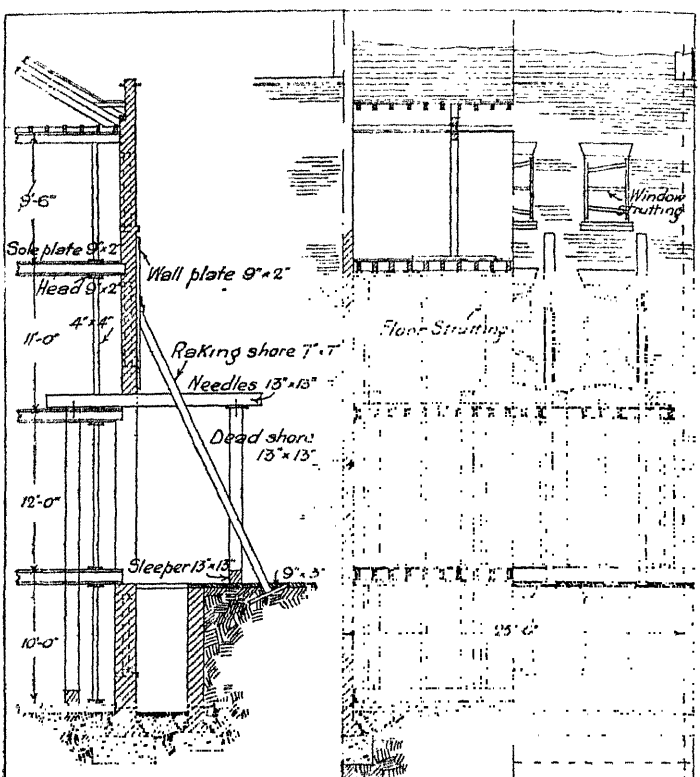


Fig. 339. Fig. 340.

Figs. 339—340.

are required to be removed, for the purpose of making large openings in the lower parts. Let the lower part of a dwelling-house be required to be removed and a shop front inserted, then the method of procedure would be as follows :—

The whole of the floors, the roof, and any other load bearing on the wall are supported by a system of strutting to relieve the wall of all weight ordinarily taken by it. This system of strutting should be firmly supported by a sole piece properly bedded on the solid ground below the basement floor. The sole plate should be bedded in mortar along its whole length and be sufficiently stiff to distribute the weight over its whole length.

Perforations are now made in the wall a short distance above the line of the top of the arch or girder that is finally to support the wall. Through the holes needles are inserted, consisting of balk timbers or steel joists; these should not be placed a greater distance than 6 feet apart in brick walls, as shown in Figs. 339 and 340.

The needles are supported by upright balks termed dead shores, one under each end of the needle. The dead shores rest at their lower ends on sleepers, horizontal balks of timber, properly bedded in mortar for their whole length. It is essential that the sleepers should be bedded on the solid ground, not on the crown of vaults or any other voids. Should there be voids of any kind the work must be solidly strutted. Pairs of hardwood wedges are placed between the tops of the dead shores and under the bearing surface of the needles; before these are driven up tightly, a bed of cement mortar should be placed on the top of the needle at the point where it passes through the wall to ensure a proper and solid bearing of the wall on the needles. When the wedges are driven home the whole is allowed a few days to set.

There is often a difficulty in getting in the dead shore on the inside in one piece; where this is the case, the lower halves are placed in first, a transom being placed across the whole of these; the upper shores are then placed on the transom directly over the lower members and under the needle at its upper end.

It is essential that the needles and dead shores should have an ample margin of strength, so that a settlement of the wall may be avoided, through deflection of the needle or compression of the shore or sole piece. The needles, shores and sleepers are well dogged together before they begin to function.

Before any of the piers are removed, all the window openings must be strutted apart, as shown in Fig. 340, to prevent any deformation taking place. In ordinary small windows this consists of an upright against each reveal, with about three struts between; but in large openings the arches require to be supported by a turning piece, or centre, made to fit, with the reveals strutted as before.

If the building be old or at all defective, raking shores are imperative, but it is wise under all conditions to use them to steady the building during the progress of the works. These are fixed against the piers between the windows and close beside the dead shores.

Having the shores all fixed in position, the two end piers should now be built, or if the supports are to be stanchions these should be erected to receive the girder, the minimum amount of the old wall being taken away to allow for this work. The intermediate piers or wall may now be removed. If an arch is to be used it should be built and the spandrel filled in to the underside of the old brickwork, or if a girder be employed it should be raised and fixed, the cover-stones bedded, and the brickwork filled in to the underside of the old work; this new brickwork should all be built in cement mortar to avoid any settlement in the new work.

A week at least should be allowed for the new work to set before any of the shoring is struck. The needles should be removed first, then the strutting from the windows, the strutting under the floors inside, and, lastly, the raking shores. About 2 days should be allowed between each of these operations in order that the work may take its bearings gradually on the new supports.

Great care is required in carrying out these operations in a corner house; the needling would be made to suit the special requirements of the case, but under all conditions the angle of the building should be shored with raking or horizontal shores if that be more convenient.

Underpinning.—It frequently happens that the lower part of a wall has to be moved either to renew defective foundations or to carry the wall down to a lower level, as in the case where a basement storey is to be constructed

under an existing house. The whole length of the wall could be carried on dead shores as previously described, but it is usual to proceed as follows. The length of the wall is divided into sections approximately 3 feet in width. Let the wall be 30 feet in length and divided into nine sections, numbered 1 to 9 from left to right. Then a suitable order of procedure would be as follows: Excavate and timber to the required depth sections 2, 5 and 8 on both sides of the wall. The concrete foundation is deposited and the section of the wall is carried up to the old work in rapid-hardening cement mortar. When these are completed, sections 3 and 7 can be proceeded with, each section of brickwork being toothed into the preceding. The remaining sections can then be excavated and built. Before commencing operations on the angle sections they should be needled to prevent any dislocation of the work at this point. By this method all timbering, except for the excavation, can be avoided.

CHAPTER XVI

RIVETING

Riveting.—All built up sections for girders or stanchions have their plates or parts connected by rivets, which are superior to bolts for this purpose, for being hammered up hot they contract on cooling, and cause a frictional resistance between the plates, and though this is not taken into account in estimating the strength of the member it adds considerably to the rigidity of the work. For the sake of rapidity in erection the connections between the members of a steel frame structure are frequently made with bolts, though riveting, by adding to the rigidity of the connections, results in a stronger and stiffer structure.

Dimensions of Rivets.—The diameter of rivets for constructional work varies from $\frac{1}{2}$ to $1\frac{1}{8}$ inch. There is no exact rule for the dimensions of rivets, but where holes are punched in the plates, practical requirements render it necessary to have the punch rather larger than the thickness of the plate. Unwin's rule $d = 1{\cdot}2\sqrt{t}$ is a good guide. A better rule for constructional work is $d = \frac{t}{8} + \frac{5}{8}$ inch, especially for members made up of several plates. In practice rivets are nearly always either $\frac{3}{4}$ or $\frac{7}{8}$-inch. diameter.

Pitch of Rivets.—The pitch of rivets is the distance measured from centre to centre. The ordinary pitches vary between 3 and 4 inches. The pitch should never exceed 6 inches or sixteen times the thickness of the thinnest plate, unless special conditions render a variation necessary. It is advisable for the sake of economy to use the same pitch throughout on any member. The pitch and arrangement of rivets is governed by the conditions for resisting failure on these points.

Rivets should be at least 3 diameters apart, centre to centre. The minimum distance from the centre of a rivet to a planed or rolled edge should be (from B.S.S. 449) :—

$1\frac{1}{2}$	inch for	1	inch	diameter	rivets.
$1\frac{1}{4}$	,, ,,	$\frac{7}{8}$	,,	,,	,,
$1\frac{1}{8}$	,, ,,	$\frac{3}{4}$	,,	,,	,,
1	,, ,,	$\frac{5}{8}$	,,	,,	,,

and to a sheared edge should be :—

$1\frac{3}{4}$	inch for	1	inch	diameter	rivets.
$1\frac{1}{2}$	,, ,,	$\frac{7}{8}$	,,	,,	,,
$1\frac{1}{4}$	,, ,,	$\frac{3}{4}$	,,	,,	,,
$1\frac{1}{8}$	,, ,,	$\frac{5}{8}$	,,	,,	,,

Holing the Plates.—Rivet holes are made in three ways : (1) Punching ; (2) Drilling ; (3) Punching and Rymering. Punching is by far the cheaper method ; it has the disadvantage of slightly injuring the plate immediately about the hole, and there is also a liability to inaccuracies in the spacing, resulting in the difficulty that when the parts are assembled the holes are not concentric. In general the punching process is frequently limited to holes of a maximum of $\frac{3}{4}$-inch diameter. Drilling, though more expensive than punching, has the advantage that it does not injure the plates, and as the parts are assembled and drilled together, absolute concentricity is obtained. The third method is a compromise between the first and second ; it consists of punching the holes about $\frac{1}{4}$ inch less than the dimension required and then enlarging the hole with the drill to its correct dimension. The holes are usually made $\frac{1}{16}$ inch larger than the shank of the rivet to be employed.

For the tests for rivets see the British Standard Specification, page 163.

Riveted joints may fail in four ways by (1) Shearing, (2) Bearing, (3) Tearing of the plate, (4) Bursting of the plate.

(1) *Shearing.*—Rivets may be in single or in double shear, according as the rivet tends to shear across one or two sections. See Fig. 341. The resistance of a rivet to shear (F_s) is the product of its sectional area into the safe shear stress (f_s) of the metal and

$$F_s = f_s \pi r^2 = f_s \frac{\pi}{4} d^2.$$

For double shear allow twice the value in single shear and

$$F_s = f_s 2\pi r^2 = f_s 2 \frac{\pi}{4} d^2$$

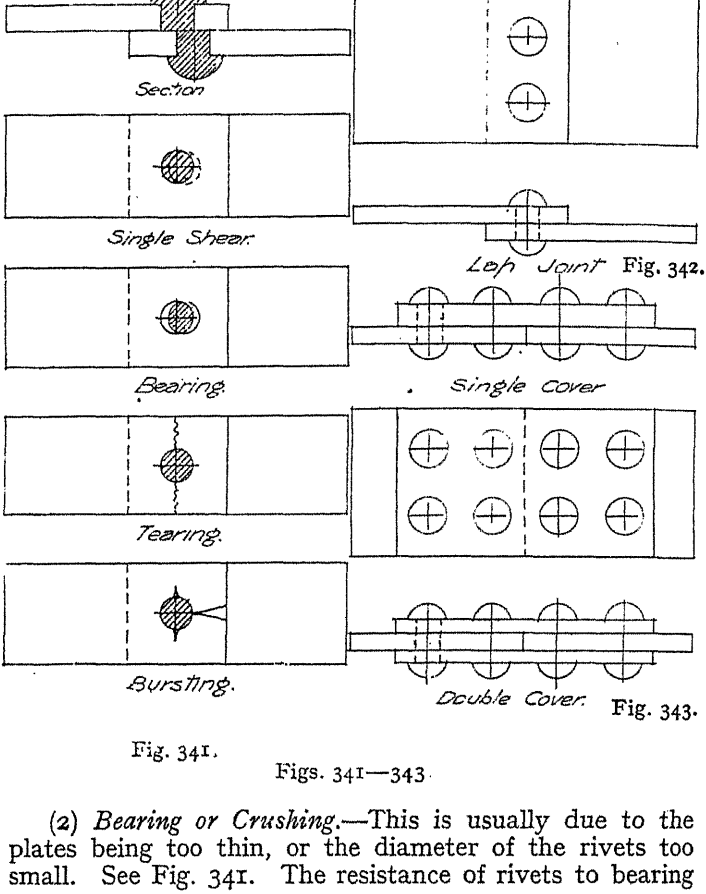


Fig. 342.

Fig. 343.

Fig. 341.

Figs. 341—343

(2) *Bearing or Crushing.*—This is usually due to the plates being too thin, or the diameter of the rivets too small. See Fig. 341. The resistance of rivets to bearing

$$F_b = f_b dt.$$

(3) *Tearing of the Plate.*—Plates connected by riveting

are weakened by the holes drilled for the rivets. The tendency in the plate to tear will always be across the rivet holes. The resistance of such a joint will be the product of the resistance of the metal into the effective sectional area of the plate.

$$F_t = f_t\ (b \times t) - (n \times d).$$

(4) *Bursting of Plate.*—This failure is likely to occur when the rivet hole is drilled or punched too near to the edge of the plate. To prevent this the hole should be at least $1{\cdot}5d$ from the centre to the edge of the plate.

Rivet Values, B.S.S.—The following table and tables on pp. 452 and 453 are made up from the section dealing with working stresses in the British Standard Specification for the Use of Structural Steel in Building, No. 449.

BLACK BOLTS (where permissible)

Table of maximum shearing and bearing values in tons.

Dia. of Rivet in Inches.	Area in Square Inches.	Maximum Shear Value.		Maximum Bearing Values at 8 tons per square inch.										
				Thickness of Plates in Inches.										
		Single Shear at 4 tons/sq. in.	Double Shear at 8 tons/sq. in.	$\frac{1}{4}$	$\frac{5}{16}$	$\frac{3}{8}$	$\frac{7}{16}$	$\frac{1}{2}$	$\frac{9}{16}$	$\frac{5}{8}$	$\frac{11}{16}$	$\frac{3}{4}$	$\frac{7}{8}$
$\frac{3}{8}$	·1104	·44	·88	·75	·94									
$\frac{1}{2}$	·1963	·79	1·57	1·00	1·25	1·50	1·75							
$\frac{5}{8}$	·3068	1·23	2·45	1·25	1·56	1·88	2·19	2·50						
$\frac{3}{4}$	·4418	1·77	3·53	1·50	1·88	2·25	2·63	3·00	3·38	3·75				
$\frac{7}{8}$	·6013	2·41	4·81	1·75	2·19	2·63	3·06	3·50	3·94	4·38	4·81			
1	·7854	3·14	6·28	2·00	2·50	3·00	3·50	4·00	4·50	5·00	5·50	6·00	7·00	
$1\frac{1}{8}$	·9940	3·98	7·95	2·25	2·81	3·38	3·94	4·50	5·06	5·63	6·19	6·75	7·88	
$1\frac{1}{4}$	1·2272	4·91	9·82	2·50	3·13	3·75	4·38	5·00	5·63	6·25	6·88	7·50	8·75	

Riveted joints may be classified as (1) Lap joints, as shown in Fig. 342 ; (2) Single Cover, as shown in Fig. 343 ; and (3) Double Cover, as shown in Fig. 343. Lap and single cover joints are not to be recommended for connecting tension plates in constructional work as the stress

FIELD RIVETS.

Table of maximum shearing and bearing values in tons.

Dia. of Rivet in Inches.	Area in Square Inches.	Maximum Shear Value.		Maximum Bearing Values at 10 tons per square inch.									
		Single Shear at 5 tons/ sq. in.	Double Shear at 10 tons/ sq. in.	Thickness of Plates in Inches.									
				¼	5/16	⅜	7/16	½	9/16	⅝	11/16	¾	⅞
⅝	·1104	·55	1·10	·94	1·17								
½	·1963	·98	1·96	1·25	1·56	1·88	2·19						
⅝	·3068	1·53	3·07	1·56	1·95	2·34	2·73	3·13					
¾	·4418	2·21	4·42	1·88	2·34	2·81	3·28	3·75	4·22	4·69			
⅞	·6013	3·01	6·01	2·19	2·73	3·28	3·83	4·38	4·92	5·47	6·02		
1	·7854	3·93	7·85	2·50	3·13	3·75	4·38	5·00	5·63	6·25	6·88	7·50	8·75
1⅛	·9940	4·97	9·94	2·81	3·52	4·22	4·92	5·63	6·33	7·03	7·73	8·44	9·84
1¼	1·2272	6·14	12·27	3·13	3·91	4·69	5·47	6·25	7·03	7·81	8·59	9·38	10·94

through the plates is not axial, and tends to bend the plates at the joint.

In the lap joint and single cover joint the rivets are in single shear, in the double cover joint the rivets are in double shear.

In designing joints, it is necessary to test for tearing, shearing, and bearing or crushing. Considerable economy may be effected by the proper disposition of the rivets. The most economical arrangement would be to get the tearing,

shearing and bearing resistances equal. For practical reasons this can rarely be attained, but the tearing and shearing can frequently be equated. The number of rivets must always depend upon which condition, shearing or bearing, gives the lesser resistance.

SHOP RIVETS AND TIGHT FITTING TURNED BOLTS.

Table of maximum shearing and bearing values in tons.

Dia. of Rivet in Inches.	Area in Square Inches.	Maximum Shear Value.		Maximum Bearing Values at 12 tons per square inch.									
		Single Shear at 6 tons/ sq. in.	Double Shear at 12 tons/ sq. in.	Thickness of Plates in Inches.									
				½	5/16	⅜	7/16	½	9/16	⅝	11/16	¾	7/8
⅝	·1104	·66	1·33	1·13	1·41								
½	·1963	1·18	2·36	1·50	1·88	2·25	2·63						
⅝	·3068	1·84	3·68	1·88	2·34	2·81	3·28	3·75					
¾	·4418	2·65	5·30	2·25	2·81	3·38	3·94	4·50	5·06	5·63			
⅞	·6013	3·61	7·22	2·63	3·28	3·94	4·59	5·25	5·91	6·56	7·22		
1	·7854	4·71	9·43	3·00	3·75	4·50	5·25	6·00	6·75	7·50	8·25	9·00	10·50
1⅛	·9940	5·96	11·93	3·38	4·22	5·06	5·91	6·75	7·59	8·44	9·28	10·13	11·81
1¼	1·2272	7·36	14·73	3·75	4·69	5·63	6·56	7·50	8·44	9·38	10·31	11·25	13·13

NOTE.—All bearing values in the preceding tables above and to the right of the upper zigzag line are greater than double shear, and therefore the shearing values govern the design. All bearing values below and to the left of the lower zigzag line are less than single shear, and therefore, in these cases, the bearing values govern the design.

The thickness of cover plates is usually determined as follows :—

$$\text{Single covers} = t + \frac{t}{8}.$$

$$\text{The sum of the thickness of double covers} = t + \frac{t}{4}.$$

EXAMPLE I.—(Fig. 344) A 10″ × $\frac{3}{4}$″ steel bar is to be jointed by a butt joint with double covers. Covers to be $\frac{1}{2}$ inch thick, rivets $\frac{7}{8}$ inch diameter, safe stress for steel in tension 8 tons per square inch, shearing 5 tons per square inch, bearing 10 tons per square inch. Determine the efficiency of the joint.

Tearing.—Then resistance (F_t) of plate to tearing, deducting one rivet hole is :—

$$F_t = (b - d)t.f_t$$
$$= (10 - \tfrac{15}{16})\tfrac{3}{4} \times 8$$
$$= 54{\cdot}3 \text{ tons.}$$

Using the appropriate table, it will be seen that for a $\frac{7}{8}$-inch diameter rivet in double shear passing through a $\frac{3}{4}$-inch thick plate, the shear strength is less than the

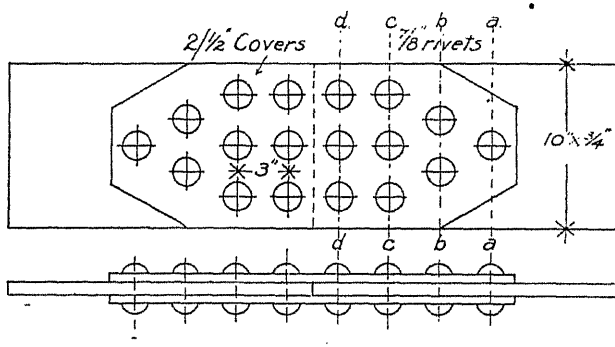


Fig. 344.

strength in bearing, and therefore the number of rivets will be determined by the strength in shear.

Shearing.—Number of rivets (n) required in double shear is

$$n = \frac{54{\cdot}3}{6{\cdot}01} = 9 \text{ rivets.}$$

If the joint were to fail, it would be across the line *a*—*a*, by the tearing of the plate, or across the line *d*—*d* by the tearing of the covers.

To establish this the resistance across the lines a—a, b—b, c—c and d—d should be determined.

This may be done as follows :—

The resistance at a—a is

$$(b - d)t.f_t$$
$$(10 - \tfrac{15}{16})\tfrac{3}{4} \times 8 = 54{\cdot}3 \text{ tons.}$$

At b—b is

$$(b - 2d)t.f_t + \text{shear resistance of one rivet.}$$
$$= \{(10 - 2.\tfrac{15}{16})\tfrac{3}{4} \times 8\} + 6{\cdot}01$$
$$= 54{\cdot}7 \text{ tons.}$$

At c—c is

$$(b - 3d)t.f_t + \text{shear resistance of three rivets}$$
$$= \{(10 - 3.\tfrac{15}{16})\tfrac{3}{4} \times 8\} + (3 \times 6{\cdot}01)$$
$$= 61{\cdot}2 \text{ tons.}$$

Along d—d the joint is liable to fail owing to the tearing of the covers.

Then the resistance of the covers is

$$(b - 3d)t.f_t$$
$$= (10 - 3.\tfrac{15}{16})\,2.\tfrac{1}{2} \times 8$$
$$= 57{\cdot}5 \text{ tons.}$$

This analysis shows that the weakest part of the joint is across the section a—a. The efficiency of a joint is the ratio of the least resistance of the joint to the resistance of the whole section of the plate.

$$\text{Efficiency} = \frac{\text{least resistance of joint}}{\text{total resistance of plate}}$$

$$= \frac{54{\cdot}3}{10 \times \tfrac{3}{4} \times 8}$$
$$= 90{\cdot}5 \text{ per cent.}$$

Rivets are frequently subjected to a shearing stress due to eccentricity of loading, in addition to the direct shear due to the load, as in the case of cleat connections, brackets, cantilevers, etc. In such cases, the connection may be assumed to tend to rotate about the centre of gravity of the rivet system. Each rivet is then subject to a shear stress at right angles to the line joining the centre of gravity and the centre of the rivet. The vertical load may be considered to be distributed evenly over all the rivets. To determine the

actual stress on the rivet, the shear due to rotation and the vertical shear must be compounded ; the resultant will be the total shear on the rivet.

The joint should be designed and the number and size of the rivets assumed. Find the centre of gravity of the rivet system, and the distance of the centre of each rivet from the centre of gravity.

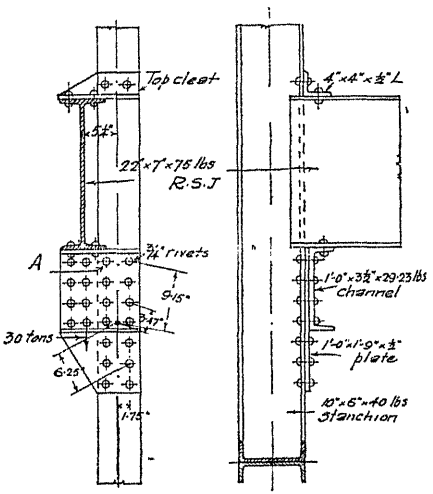


Fig. 345.

The resistance of each rivet to rotation is obtained from the formula:—

$$\frac{M}{I} = \frac{f}{y}$$

where M is the bending moment obtained by multiplying the total load carried by the amount of the eccentricity of the centre of gravity of the rivet system.

I is the moment of inertia of the rivet system about its centre of gravity.

y is the distance of the centre of the extreme rivet from the centre of gravity of the rivet system.

f is the stress in the extreme rivet.

EXAMPLE.—A rolled steel beam 22″ × 7″ × 75 lbs. is carried at the side of a 10″ × 6″ × 40 lbs. stanchion. The bracket is formed of a 1′ × 1′ 9″ × ½″ plate and 12″ × 3½″ × 29·23 lbs. channel 1 foot long. The bracket is connected to the stanchion by 14 — ¾ inch diameter rivets. The centre of the beam is 5¼ inches from the centre of gravity of rivet system. The reaction of the beam is 30 tons. For further dimensions, see Fig. 345.

Determine the total shear on the most distant rivets.

The bending moment M is,

$$M = 30 \times 5{\cdot}25 = 157{\cdot}5 \text{ tons inches.}$$

From Fig. 345, $y = 9{\cdot}15$ inches.

Now the moment of inertia of an area about any point is obtained by taking a number of small areas over the main area, multiplying each by the square of its respective distance, from the point about which the inertia is being calculated, and finally adding these products together. Thus,

$$I = \Sigma a y^2.$$

In this case the moment of inertia of the rivet system can be obtained by letting a be the area of a rivet and y the varying distances of the rivets from the centre of gravity of the rivet system.

Then

$$\begin{aligned} I &= a\{(4 \cdot 9{\cdot}15^2) + (4 \cdot 6{\cdot}25^2) + (4 \cdot 3{\cdot}47^2) + (2 \cdot 1{\cdot}75^2)\} \\ &= a\{335 + 156 + 48{\cdot}2 + 6{\cdot}1\} \\ &= 545{\cdot}3 \cdot a \text{ inches}^4 \end{aligned}$$

$f = \frac{My}{I}$. Let F = total shear load in extreme rivet due to rotation.

Then $F = f \cdot a$ or $\frac{F}{a} =$

Hence $\frac{F}{a} = \frac{My}{I}$

or $\frac{F}{a} = \frac{157{\cdot}5 \times 9{\cdot}15}{545{\cdot}3\ a.}$

$$F = \frac{157{\cdot}5 \times 9{\cdot}15}{545{\cdot}3}$$

$$= 2{\cdot}64 \text{ tons.}$$

Vertical load on rivets is :

$$\frac{30}{14} = 2{\cdot}14 \text{ tons.}$$

Consider Fig. 346. Let G be the centre of gravity of the rivet system and A the position of one of the extreme rivets. Let AE be the vector of the shear load on rivet due to rotation (2·64 tons), and AC be the vector of the vertical shear load (2·14 tons). Then AB is the resultant total shear load on this rivet.

$$\text{Angle AGH} = \tan^{-1}\frac{9''}{1{\cdot}75''}$$

$$= 79°$$

$\therefore$ Geometrically, angle DCB = 79°

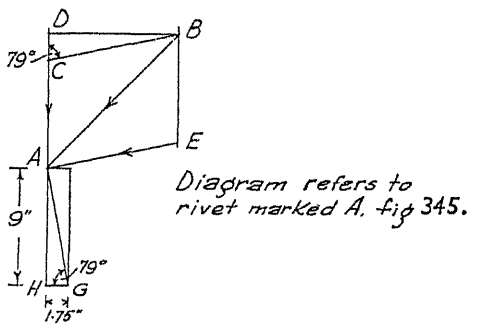


Fig. 346.

Then the resultant of the vertical and rotary shear may be determined by the formula for obtaining the resultant, p. 464.

$$R = \sqrt{2{\cdot}14^2 + 2{\cdot}64^2 + 2 \,.\, 2{\cdot}64\,(2{\cdot}14 \,.\, \cos 79°)}$$

$$= 3{\cdot}7 \text{ tons.}$$

At 6 tons/square inch the value given in the table for a $\frac{3}{4}$-inch diameter rivet in single shear is 2·65 tons. Therefore the value is high, being 8·38 tons/square inch as against the allowable 6 tons/square inch.

With the angle cleat at the top of the beam, as shown in Fig. 345, the arrangement would be satisfactory. The other rivets would be within the prescribed amount. The

allowance for bearing for a $\frac{3}{4}$-inch diameter rivet in a $\frac{1}{2}$-inch plate at 12 tons/square inch is 4·5 tons.

EXAMPLE.—A skeleton cantilever formed as shown in Fig. 347 is connected to a stanchion formed from two 10″ × 3½″ × 24·5 lbs. channels by a $\frac{5}{8}$-inch plate. The latter is secured between the channels by eighteen $\frac{7}{8}$-inch diameter rivets arranged as shown on Fig. 347. Determine the stress on the plate and rivets.

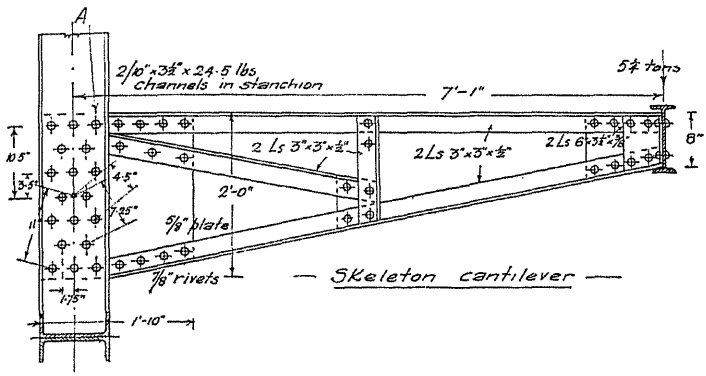


Fig. 347.

Bending moment on plate $= 5{\cdot}25 \times 85 = 446$ tons inches.

Section modulus (Z) of $\frac{5}{8}$″ plate $= \dfrac{I}{y}$

$$= \frac{bd^3}{12y}$$

$$= \frac{5 \times 24^3}{8 \times 12 \times 12} = 60 \text{ inches}^3.$$

Then

Stress in plate (f) is

$$f = \frac{M}{Z} = \frac{446}{60} = 7{\cdot}4 \text{ tons/sq. inches.}$$

Safe allowable stress for steel is 8 tons/sq. inch.

Determine total shear on one of the four extreme rivets, say, A.

Then, as before, moment of inertia of rivet system is

$$I = a\{(4 \cdot 11^2) + (2 \cdot 10{\cdot}5^2) + (4 \cdot 7{\cdot}25^2) + (4 \cdot 4{\cdot}5^2) + (2 \cdot 3{\cdot}5^2) + (2 \cdot 1{\cdot}75^2)\} = 1027 \cdot a \text{ inches}^4.$$

Then $f = \frac{My}{I}$. Let F = total shear load on extreme rivet due to rotation.

$$\therefore \quad = \frac{F}{a}.$$

Substituting $F = \frac{Mya}{I} = \frac{446 \times 11 \times a}{1027 \cdot a}$

$$F = 4{\cdot}78 \text{ tons.}$$

In addition vertical load on rivet is

$$\frac{5{\cdot}25}{18} = 0{\cdot}29 \text{ tons.}$$

For the total load on extreme rivet, compound the load due to rotation and the vertical load.

Consider Fig. 348. Let G be the centre of gravity of the rivet system and A the position of one of the extreme rivets. Let AE be the vector of the shear load on rivet due to rotation (4·78 tons), and AC be the vector of the vertical shear load (0·29 tons). Then AB is the resultant total shear load on this rivet.

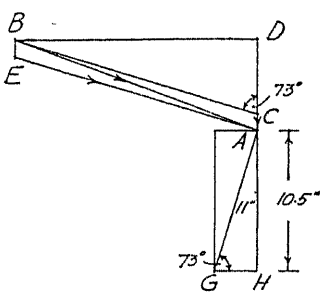


Fig. 348.

$$\text{Angle AGH} = \sin^{-1}\frac{10{\cdot}5}{11}$$

$$= 73°.$$

∴ Geometrically, angle DCB = 73°

The resultant of the vertical and rotary shear may be determined as before.

$$AB^2 = AC^2 + AE^2 + 2 \cdot AC \cdot CB \cos 73°$$

$$R = \sqrt{0{\cdot}29^2 + 4{\cdot}78^2 + 2 \cdot 0{\cdot}29 \cdot 4{\cdot}78 \cdot \cos 73°}$$

$$= 4{\cdot}87 \text{ tons.}$$

At 5 tons/square inch the value given in the table for a $\frac{7}{8}$-inch diameter rivet in double shear is 6·01 tons, and a bearing value in a $\frac{5}{8}$-inch plate of 5·47 tons.

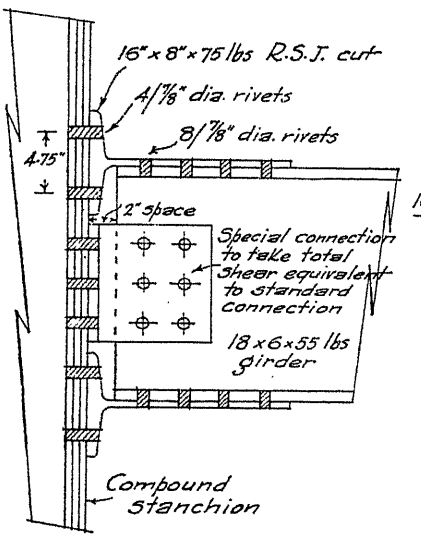


Fig. 349.

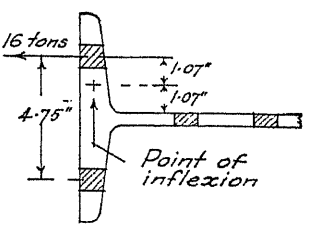


Fig. 350.

EXAMPLE.—Fig. 349 shows a type of rigid connection used when a beam is subject to a large end moment due to the stresses set up by wind pressure. In making the calculation all the working stresses can be increased by 33⅓ per cent. (see B.S.S. No. 449).

Let the girder be 18" × 6" × 55 lbs., and let the end moment set up by the wind be 550 tons inches. This moment is well within the capacity of the beam. The connection consists of a special web connection designed to carry the reaction of the beam, and, top and bottom, a

section of 16″ × 8″ × 75 lbs. R.S.J. with one flange cut away. The width of the stanchion and length of the 16″ × 8″ tee would be a mimimum of about 14 inches. Allow eight $\frac{7}{8}$-inch diameter rivets in single shear through the leg of the tee and eight $\frac{7}{8}$-inch diameter rivets through the flange.

At 5 tons/inches.^{2} Value of 8 rivets in single shear

$$= 8 \times 3{\cdot}01 \times 1{\cdot}33 \text{ (wind)}$$
$$= 32 \text{ tons.}$$

Consider $\frac{32}{2} = 16$ tons as carried by the top row of rivets through the flange.

Calculate the bending moment and necessary thickness of the tee.

Referring to Fig. 350, the bending moment set up on the flange of the tee is

$$16 \times 1{\cdot}07 = 17{\cdot}1 \text{ tons inches.}$$

Assume connection to be 14 inches long.

Now $$\frac{M}{I} = \frac{f}{y}$$

and $$I = \frac{bt^3}{12}, \text{ also } y = \frac{t}{2}.$$

$\therefore$ Substituting $$t = \sqrt{\frac{6M}{fb}}$$

$$f = 8 \text{ tons/inches}^2 \times 1{\cdot}33 = 10{\cdot}65 \text{ tons/inches}^2 \text{ (for wind)}$$

$$\therefore \quad t = \sqrt{\frac{6 \times 17{\cdot}1}{10{\cdot}65 \times 14}} = 0{\cdot}83 \text{ inches.}$$

As the thickness of the connection is 0·94, the strength of the tee is sufficient to take the bending moment set up by capacity value of the eight rivets in single shear, namely, 32 tons.

Using an 18″ × 6″ × 55 lbs. girder the end moment is then: $32 \times 18 = 570$ tons inches, which is sufficient for the case in hand.

CHAPTER XVII

GRAPHIC STATICS

Calculations.—The calculation of buttresses, retaining walls, arches, girders, trusses, and such constructions which resolve themselves into statical problems, may be accomplished in two ways: (1) By pure mathematics; (2) by graphic statics.

The first method gives results correct to any required number of decimal places.

The graphic method gives results varying according to the accuracy and sensibility of the mathematical drawing instruments used, together with the carefulness and skill of the operator; and provided these conditions are absolutely satisfied, correct results would be obtained in a very short time. The diagram (if mathematical accuracy is desired) should be drawn with units, the lengths of which are not less than a thousand times the thickness of the lines employed; and with good instruments, judgment and care, the answer would probably be correct to within a one-thousandth part of itself, which is quite satisfactory for ordinary practice, and added to this any error in the equilibrium of the forces is evident by inspection of the diagram, which makes this method invaluable as corroborative evidence, even though trigonometrical calculations are used.

It would probably be true to say that in ordinary practice answers would be obtained by the graphic method correct, or with a maximum error of one per cent., and when it is remembered that for safety the strength of parts is made from four to ten times the calculated breaking strengths, an error of one per cent., although undesirable, is practically immaterial.

The magnitude and direction of any force may be represented by a line drawn to scale. The sense is indicated by figures, letters or arrowheads. The directed quantity thus represented is known as a Vector. When several vectors act at a point or on a body, the effect of the whole may be represented by a single force known as the Resultant, or their effect may be neutralised by a single force, equal to the resultant in magnitude but opposite in direction, known as the Equilibrant.

Where two forces AE and AC (as shown in Figs. 351 and 352) meet at a point, the resultant may be found by drawing a line parallel to each force from the free end of the other to form a parallelogram. Then the diagonal AB drawn through the point of application A to B will be the resultant of the two given forces.

Let AC, AE be the given vectors, from E and C draw CB and EB parallel to AC and AE respectively. Then the diagonal AB will represent the resultant. The proof of this statement is given in Euclid II., 12 and 13, and may be expressed algebraically as follows :—

$$AB^2 = AC^2 + CB^2 \pm 2AC.CD.$$

The addition or subtraction of the last expression depends upon whether the angle included between the two vectors is acute or obtuse (see Figs. 351 and 352).

In a similar manner, any number of forces meeting at a point may be compounded by combining any two of them to find their resultant, then combining the latter with the next force in the system to find a second resultant, and so on, till the last force has been compounded.

If the sense of the resultant be reversed, it becomes the equilibrant, or that force that will produce a state of equilibrium.

In Fig. 353 since CB = AE in magnitude and direction, CB may be substituted for AE in determining either the resultant or equilibrant.

If AB, BC, CA represent three forces in equilibrium, meeting at a point, their relative magnitudes may be determined by drawing parallels to each of them so that the arrows representing their directions are concyclic, when they will form a triangle. Thus if the direction of each is

given, and the magnitude of one of them, the magnitudes of the remaining two may be found. The triangle is termed the Reciprocal of the forces.

If the reciprocal to any system of forces, when drawn, does not form a closed figure, it indicates that the system is not in equilibrium. The line necessary to close the figure will represent in magnitude and direction the vector required to produce equilibrium.

Similarly, where a number of forces act at a point or on a body, the method of drawing a reciprocal figure may be employed to determine if they are in a state of equilibrium, or what additional force is required to produce a state of rest. The reciprocal in this case is termed the polygon of forces. See Figs. 354 and 355. The proof of this statement may be enunciated by referring the system of forces to two axes XOX′ and YOY′ at right angles to each other, the point of application being at the origin O (Fig. 356). Then the condition to be satisfied is that the sum of the components of all the forces parallel to XOX′ or YOY′ shall be zero.

Thus the components of any force F making any angle with XOX′ will be

$$\begin{matrix} \text{F cos } \theta, \text{ F sin } \theta \text{ parallel to} \\ \text{XOX′ or YOY′ respectively.} \end{matrix}$$

Hence for the equilibrium we must have

$$\left.\begin{matrix} \Sigma \text{ F cos } \theta = \text{O} \\ \Sigma \text{ F sin } \theta = \text{O} \end{matrix}\right\}.$$

If the forces are not in equilibrium

$$\left.\begin{matrix} \Sigma \text{ (F cos } \theta) = x \\ \Sigma \text{ (F sin } \theta) = y \end{matrix}\right\}.$$

Then if the resultant be denoted by R and act at O in a direction making an angle θ with XOX′ we have

$$\text{R} = \sqrt{x^2 + y^2} \tag{1}$$

$$\tan \theta = \frac{y}{x} \tag{2}$$

x = the sum of the resolved parts of the forces parallel to XOX′ and

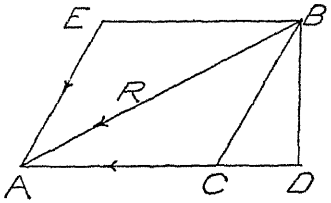


Fig. 351.

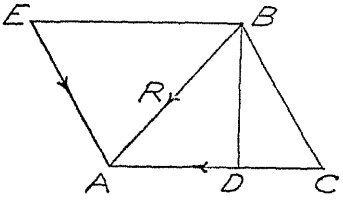


Fig. 352.

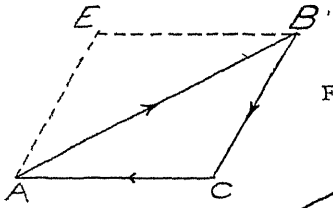


Fig. 354.

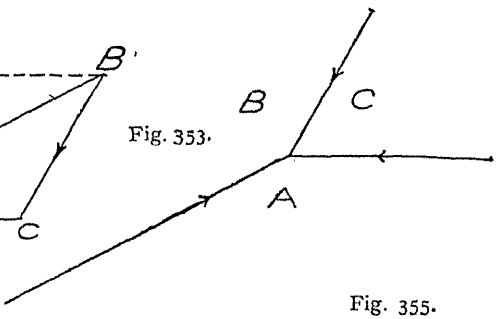


Fig. 353.

Fig. 355.

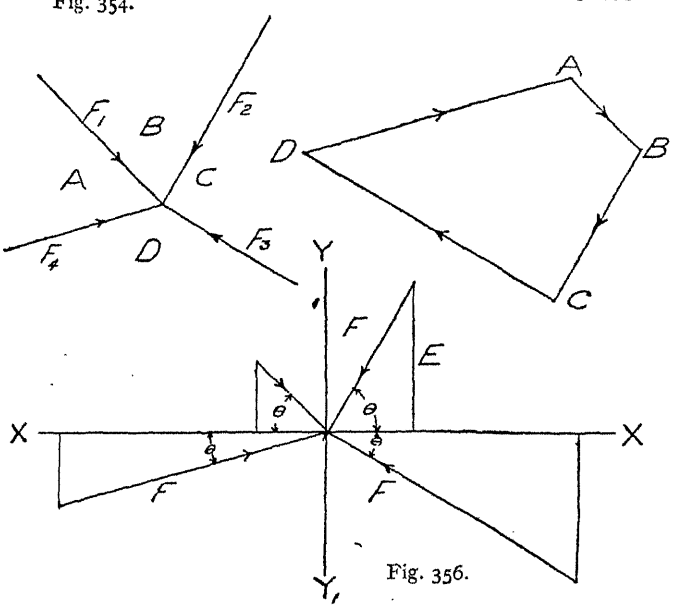


Fig. 356.

Figs. 351—356.

y = the sum of the resolved parts of the forces parallel to YOY′.

Where a group of coplanar forces acts on a body but does not meet at a point, the resultant or equilibrant may be determined in magnitude, direction and position by the

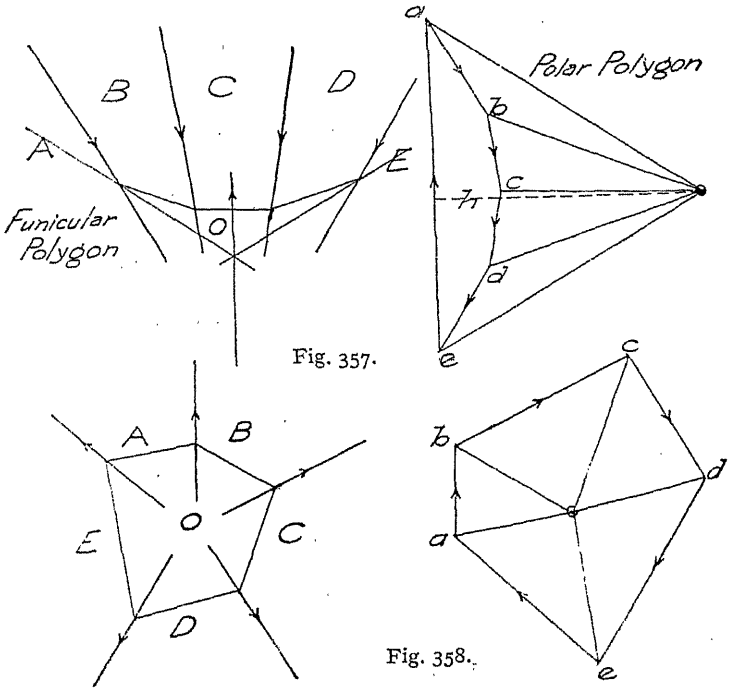


Fig. 357. Fig. 358.

Figs. 357—358.

method of the Link or Funicular polygon (Figs. 357 and 358).

Let AB, BC, CD, DE be four such forces. Proceed as in the previous case by constructing a reciprocal polygon, by drawing parallels to the given forces; if the resulting reciprocal forms a closed polygon it indicates that the forces are in equilibrium. If as in Fig. 357 they do not close

the polygon, then the line *ea* required to complete the figure is the equilibrant or the resultant according to its sense. The magnitude and direction of the resultant having been found, it remains to determine its position relative to the given forces.

Select a point O (Fig. 357) in a convenient position, from this point, termed the Pole, draw lines to each of the points of the force or reciprocal polygon, then draw a parallel to *ao*, passing anywhere through the force AB or AB produced ; from the point of intersection with AB draw a parallel to *bo* across the space B to cut BC, from this point draw a parallel to *co* across the space C, and so on till the last parallel cuts E ; from this point draw a parallel to *eo* and produce it till it intersects the projection of AO. Through the point of intersection draw a parallel to the equilibrant EA. This determines the position of the equilibrant.

The preceding may be demonstrated as follows : In drawing the polar polygon the pole O may be selected arbitrarily in any convenient position. Take any nucleus of forces from the funicular polygon, say AB, BO, OA. These three forces are in equilibrium because their reciprocals *ao*, *ob*, *ba* on the polar polygon form a closed triangle of force *abo*, the lines *ao*, *bo* giving the magnitudes of the two components of AB on the funicular polygon. Similarly in the nucleus BC, CO, OB, their reciprocals on the polar polygon also form a closed triangle, having one side in common with the triangle *abo*, and similarly till the last nucleus EA, AO, OE is reached, and as *ea* is the resultant of all the other forces it forms with the lines *ao*, *oe* a triangle of forces in equilibrium, having two of its sides coinciding with two other triangles in the system.

It will also be evident that if the links of the funicular polygon be hinged they would remain in equilibrium while the system of forces AB, BC, CD, DE and EA acted in the position shown.

Any number of other link systems could be arranged about the system of forces, the condition for equilibrium being that the system of links should form a closed polygon. The reciprocal of any of these link systems will form a polygon with a common point O the pole, the reciprocal of

the links being drawn from the extremities of the reciprocals of the systems of forces.

In a polar polygon, the moment of the resultant about the pole is equal to the sum of the moments of all the other forces about the same pole. It follows from this, that any force in a system in equilibrium may be considered as the resultant or equilibrant of the remainder according to its sense, since the removal of any would disturb the equilibrium of the remainder.

As in the last case for a series of forces in equilibrium meeting at a point

$$\Sigma \text{ F} \cos \theta = \text{O}$$
$$\Sigma \text{ F} \sin \theta = \text{O}$$

Denoting the vertical and horizontal components of the resultant

$$y = \Sigma \text{ F} \sin \theta \text{ and } x = \Sigma \text{ F} \cos \theta \text{ respectively,}$$

$$\text{R} = \sqrt{x^2 + y^2},$$

and its inclination to the XOX axis is

$$\tan \theta = \frac{y}{x}.$$

Moments.—The moment of a force is its tendency to cause rotation about a point, and is equal to the product of a force multiplied by its perpendicular distance from the point.

To determine the moment of any number of forces about a given point P in the plane. Let AB, BC, CD, DE represent the given forces (Fig. 359). Draw the reciprocal figure *ab, bc, cd, de* and close the polygon by joining the points *ea*. Take any pole and draw the polar polygon *oa, ob . . . . oe* and from this, the corresponding funicular polygon, by drawing parallels commencing at any point, say Z in AB. Then the intersection of the links *Zm* and *qn* will be a point in the pathway of the resultant, the magnitude will be given by the line *ea* on the force polygon. The resultant R may be drawn through the point *n*. To determine the moment of R about the point P and therefore of the four forces AB, BC, CD and DE about P produce the link *Zm*, and draw

a parallel to R through the point P cutting Zm and qn in m and l. Draw the line x, the leverage of R and a parallel through O in the polar polygon to cut ea in h. Then $R \times x = y \times oh$ the moment of the given forces about P.

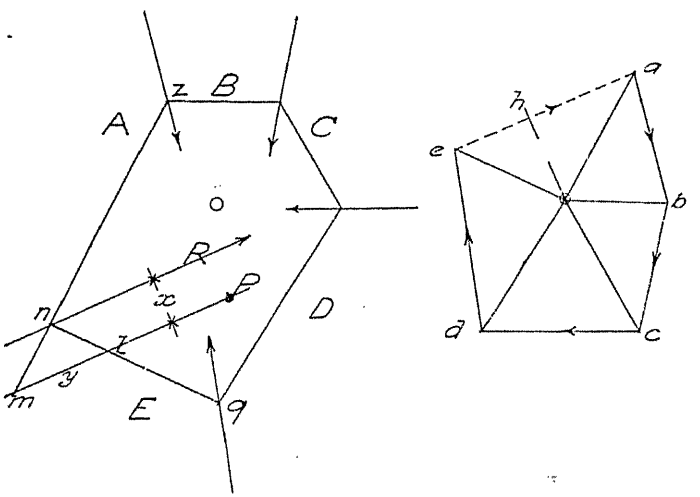


Fig. 359.

Proof.—The triangles lmn on the funicular and oea on the polar polygon are similar

$$\text{and } \frac{ml}{x} = \frac{y}{x} = \frac{ea}{oh}$$

$$\text{that is } y \times oh = ea \times x.$$

In the particular case of Parallel forces and two equilibrants, as in a beam, the preceding construction may be employed to determine the reactions and the moment at any point in the beam. Let AB, BC, CD (Fig. 360) be three parallel forces on the beam and AE, ED the reactions of the supports. Draw the force polygon ab, bc, cd. Then the line drawn from d to $a = AE + ED$; to determine their separate values, construct a polar polygon, for the purpose of obtaining the moment at any point in the beam, place

the pole O a convenient number of units, say 10, from the resultant *ad*. Construct the funicular polygon and complete it by drawing the closing line from the points of intersection of the parallels to *ao*, *do*, with AE and DE. From the pole, draw a parallel to cut the resultant in *e*, then *de* and *ea* will give the values of the respective reactions DE, EA.

To determine the moment of the loads on the beam, about any point *x*, measure the intercept *y* drawn perpen-

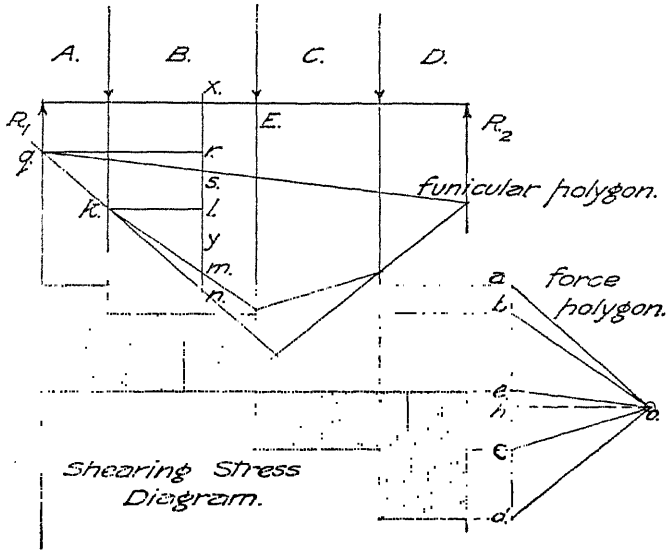


Fig. 360.

dicularly below *x*, and multiply this by *oh*, *i.e.*, $y \times oh =$ moment on the beam.

Proof.—In Fig. 360 the triangles *qsn* and *aeo* are similar, and *qr* is parallel to *oh*

$$\therefore \frac{sn}{qr} = \frac{ae}{oh}$$

$$sn \times oh = ae \times qr$$

but $ae \times qr$ = moment of AE about x.
$\therefore sn \times oh$ = AE $\times$ x the moment at x,
but the force AB is opposite to AE,
and the triangles kmn and abo are similar,
and kl is parallel to oh

$$\therefore \frac{mn}{kl} = \frac{ab}{oh}$$

$mn \times oh = ab \times kl$,
but $ab \times kl$ = moment of AB about x.
$mn \times oh =$ " "
Then the moment at $x = (sn - mn) \times oh$
$= y \times oh$.

Hence the bending moment at any point along the beam equals the intercept on the bending moment diagram at that point multiplied by the distance *oh*. The intercept is measured to the same scale as the load scale in the force polygon and the polar distance is measured to the same scale as that to which the span of the beam is drawn.

To determine Deflection Graphically.—To design a beam to carry a distributed load varying from zero at one end to a maximum of 10 tons per foot run at the other, and to find the maximum deflection. Span of beam to equal 15 feet.

Draw to scale a load diagram and divide into a number of strips, say at 1 foot intervals, as shown by the firm lines. Consider the portions of load represented by these strips to be acting as concentrated loads down the dotted lines and to be proportional in magnitude to the length of the dotted line. We have now transformed the distributed load into fifteen concentrated loads. Number these forces in the conventional fashion. Construct the polar polygon and hence get the bending moment diagram.

To determine a suitable R.S.J. we must determine the maximum bending moment. From the drawing this occurs at a point 8 ft. 6 in. from the right-hand support.

Intercept on B.M. diagram = 1·63″
Polar distance = 2″

Then, as stated above,

Maximum B.M. = (intercept to load scale) × (polar distance to linear scale).
= (1·63 × 16) (2 × 2·66)
= 138·8 tons ft.
= 1665 tons in.

Use the formula

$$\frac{M}{f} = Z \text{ (see page 554)}$$

$$f = 8 \text{ tons/square inch.}$$

$$\therefore Z = \frac{1665}{8} = 208$$

Referring to the table for British standard beams for this value of Z it is found that a suitable beam is a compound of 1 No. 18″ × 6″ × 55 lbs. with 1 No. 10″ × $\frac{7}{8}$″ plates on each flange. Moment of Inertia of this beam is 2,152 inches.[4]

Now the deflection diagram bears to the bending moment diagram the same relation that the bending moment diagram bears to the load diagram (Fig. 361).

Compare

$$\frac{d^2M}{dx^2} = w \text{ and } \frac{d^2y}{dx^2} = \frac{M}{EI} \text{ (see page 563).}$$

Hence proceed in exactly the same manner to draw a second polar polygon and thence a second funicular polygon. The dotted lines on the B.M. diagram represent in length, as before, the area of the strip which they bisect. The distances, 1–2, 2–3, etc., on the polar polygon are proportional to the length of the corresponding dotted lines, and hence the total length 1–16 represents the total area of the B.M. diagram.

From this latter fact the scale of the second polar polygon is obtained. In this case

1″ = 2 square inches of B.M. diagram.

Now

1″ vertical on B.M. diagram = (1″ × 16 tons) (2″ × 2·66)
1″ horizontal on B.M. diagram = 2·66 ft.
∴ 1 square inch of B.M. diagram = 16 × 2 × 2·66² tons ft.²

The intercept on the second funicular polygon represents the deflection at any point of the beam multiplied by EI.

To obtain this deflection proceed as before. Measure the intercept to the vertical scale of the polar polygon and multiply by the polar distance, *i.e.*, 2 inches, measured to the linear scale.

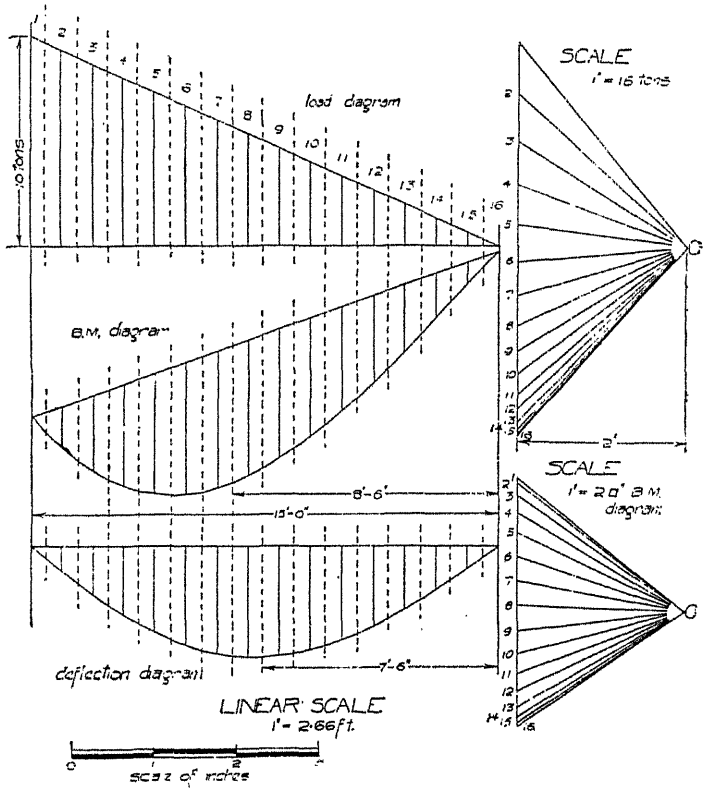


Fig. 361.

From the diagram the point of maximum deflection seems to occur at a point 7 ft. 6 in. from the right-hand support.

Intercept at this point = 1·32"
Polar distance = 2"

Scale of polar polygon
1″ = 2 square inches B.M. diagram
= 2 × (16 × 2 × 2·66²)
∴ EIy = (1·32 × 2 × 16 × 2 × 2·66²) × (2 × 2·66) tons ft.³

∴ y (in inches) = $\frac{1{\cdot}32 \times 8 \times 16 \times 2{\cdot}66^3}{29{,}000{,}000 \times 2152} \times 2240 \times 12 \times 12 \times 12$ in.

∴ $y = 0{\cdot}20''$

The exact deflection in this case has been obtained mathematically (see p. 582).

The Centre of Gravity or Centroid of an area may be

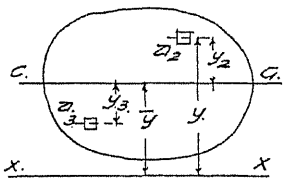


Fig. 362.

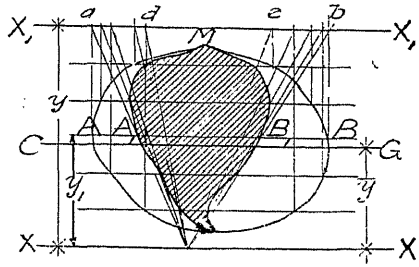


Fig. 363.

Figs. 362—363.

defined as that point about which the sum of the moments of all the particles composing the area equal zero.

$$\Sigma y\, \delta a = 0.$$

Where δa = a small particle of area situated at a distance y from the axis through the centroid.

Let the given Fig. 362 be a certain area, XX an axis drawn in any convenient position, and "a_2" an element of that area distant y from XX.

Then a_2y = moment of element about XX. Let the sum of all the elements a_2 = A = area of the whole figure, then $\Sigma a_2y = A\bar{y}$ = the moment of the whole area about XX, $\bar{y}$ being the distance of the line passing through the CG parallel to XX.

$$\text{Then } \bar{y} = \frac{\Sigma a_2 y}{A}.$$

If the line CC passes through the CG, a and a_2 are elements on opposite sides of the line, if the moment be considered positive, the other will be negative, and the sum of the moments of all such elements on one side will equal the sum of the moments of all the similar elements on the other side.

$$\text{That is } \Sigma a_2 y_2^2 + \Sigma a_3 y_3^2 = 0.$$

The centroid of any figure may be determined by the following method if the area of the figure can be ascertained. The area of any figure, regular or irregular, can be ascertained with the aid of a planimeter.

Let ABM (Fig. 363) be the given figure. Draw two parallel axes XX and X_1X_1 one on each side of the figure. Divide the figure into a series of narrow strips or laminæ by lines parallel to XX such as AB, etc. From the extremities of these lines draw parallels preferably perpendicular to X_1X_1, meeting it in *ab*, *de*, etc. From these points draw lines converging on to a pole O selected anywhere on XX. Join the points at the intersection of the converging lines with the lines representing the corresponding laminæ, producing the shaded figure A_1B_1M, termed the equivalent figure.

The first moment of the given figure about the line XX will be the product of the shaded figure into the distance y,

$$A_1 \times y.$$

Proof.—From the given figure the triangles OA_1B_1, Oab are similar, and if the line $AB = m$ and $A_1B_1 = m_1$,

$$\text{then } \frac{m}{m_1} = \frac{AB}{A_1B_1} = \frac{y}{y_1}$$

$$\text{and } my_1 = m_1 y.$$

Then find the summation Σmy_1 that is, multiply each strip by its distance y_1 and add each of these products; this will equal the product of the whole area A into the distance of its centroid $\bar{y}$ from XX, *i.e.*,

$$\Sigma my_1 = A\bar{y} = \Sigma m_1 y.$$

In the summation of $\Sigma m_1 y$, y is constant, and $\Sigma m_1 = A_1$ the mass of the shaded area,

$$\text{then } A\bar{y} = A_1 y$$

$$\text{and } \bar{y} = \frac{A_1 y}{A}.$$

A and A_1 can be determined by the planimeter, y is known, therefore, $\bar{y}$ the distance of a line passing through the centroid and parallel to XX can be determined. To find the actual centroid, it would be necessary to repeat the

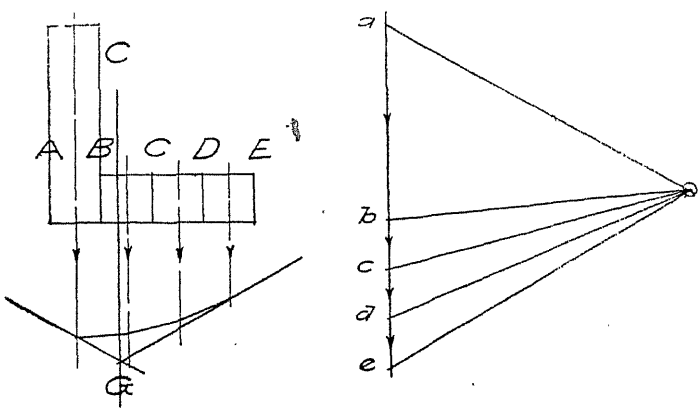


Fig. 364.

process, using two other axes at a convenient angle to XX, but not necessarily at right angles.

The simplest method of determining the centre of gravity of any figure is that of the funicular polygon (see Fig. 364).

Divide the area up into a number of parallel strips of equal width, draw a line through the centre of each strip; these centre lines on any homogeneous substance will then be proportional to the areas of the strips and may be considered to be a series of parallel forces. Draw a force polygon *a*, *b*, *c*, *d*, *e*, reciprocal to the given forces, and from this draw the corresponding funicular. Then the resultant drawn through the intersection of the

reciprocal of *ao* and *eo* produced will be a line or axis passing through the CG. If now the figures be divided into another series of parallel strips in any direction not parallel to the first series of strips, but preferably at right angles to them, then the resultant obtained by repeating the process will intersect the first resultant, the point of intersection of the two resultants will be the CG.

The Moment of Inertia or second moment of an area about any axis is the product of all the elements of the area multiplied by their distances squared from the given axis.

Thus $I_{CG} = \Sigma y^2 a$.

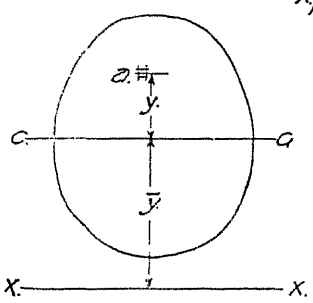


Fig. 365.

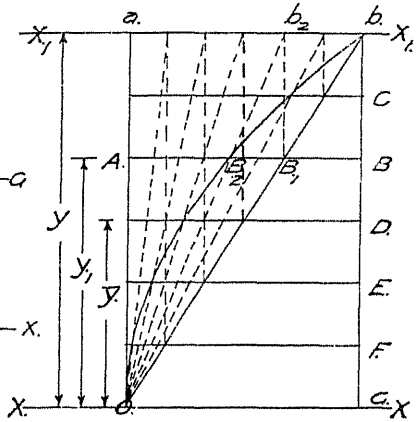


Fig. 366.

Figs. 365—366.

The moment of inertia about any axis XX not passing through the CG is

$$I_{xx} = I_{CG} + A\,\bar{y}^2.$$

Let (Fig. 365) be the given area, XX the given axis, CG the line passing through the centre of gravity, "a" an element of the area, A the whole area.

$$\begin{aligned} \text{Then } I_{xx} &= \Sigma a\,(y + \bar{y})^2 \\ &= \Sigma a\,(y^2 + 2y\bar{y} + \bar{y}^2) \\ &= \Sigma ay^2 + \Sigma a\,2y\bar{y} + \Sigma a\bar{y}^2. \end{aligned}$$

Then $\Sigma ay^2 = I_{CG}$

$\Sigma a\, 2y\bar{y} = 2\bar{y}\,\Sigma ay$

$\Sigma ay = 0$ = the 1st moment of the area about a line passing through the CG

$\Sigma a\bar{y}^2 = A\bar{y}^2$

$\therefore\ I_{xx} = I_{CG} + 0 + A\bar{y}^2$

and $I_{CG} = I_{xx} - A\bar{y}^2$

To determine the centroid and moment of any figure. Let the rectangle (Fig. 366) Oab_1G be the given figure, divide it into a series of strips parallel to XX, select a pole on XX preferably at O in the position shown. Project the extremities of the laminæ CBDEF on to the line X_1X_1; in this case they will all project into the point b_1. Draw the line b_1O from b_1 to O, this represents the common polar line of the laminæ CBDEF; the points of intersection of this line b_1O with the various strips will give points in the outline of the equivalent figure; in this case the latter is the triangle ab_1O.

Then the area of the triangle Oab_1 multiplied by y equals the first moment of the area Oab_1G about XX.

Now from page 477

$$\bar{y} = \frac{A_1 y}{A}$$

Then as $A_1 = \frac{A}{2}$

Then $\bar{y} = \frac{y}{2}$

and A_1y is the first moment of the figure about XX.

To determine the second moment of the figure about XX, the preceding construction is repeated, a second derived figure, in this case a parabola, being produced from the first.

Then, if the line $AB_2 = m_2$, the lamina m_2 of the second derived figure will be to the lamina m_1 of the first derived figure in the ratio $\frac{y_1}{y}$,

$$\text{and } m_2 = m_1\frac{y_1}{y} = m\frac{y_1^2}{y^2}$$

$$\text{since } m_1 = \frac{my_1}{y}$$

Then

$$m_2y^2 = my_1^2$$
$$y^2\Sigma m_2 = \Sigma my_1^2 = I_{xx}$$
$$A_2y^2 = Ay_1^2$$
$$\therefore A_1y^2 = I_{xx}$$

To determine the moment of inertia about a line CG parallel to XX.

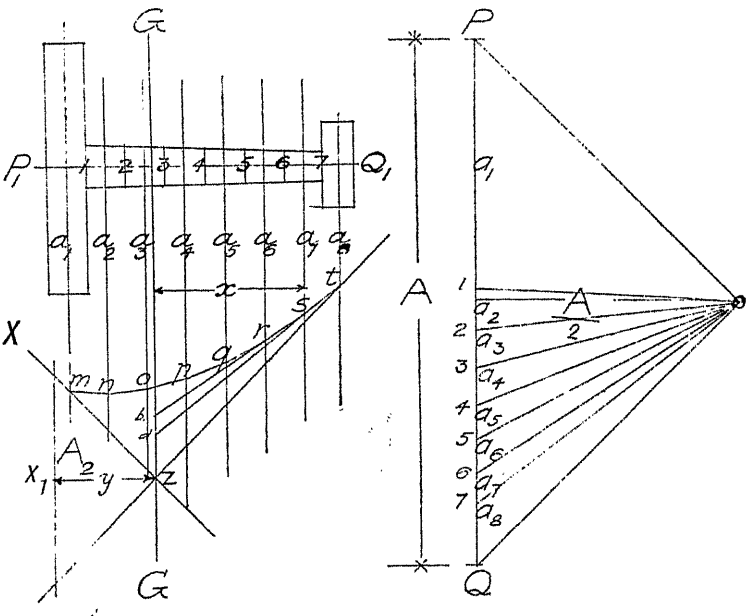


Fig. 367.

From page 479 $I_{cg} = I_{xx} - A\bar{y}^2$

$$= A_2y^2 - A\bar{y}^2$$

Since $\bar{y} = \frac{A_1y}{A}$ $\qquad \bar{y}^2 = \left(\frac{A_1}{A}\right)^2 y^2$

$$\therefore I_{cg} = A_2y^2 - A\left(\frac{A_1}{A}\right)^2 y^2$$

$$= y^2\left(A_2 - \frac{A_1^2}{A}\right).$$

The moment of inertia can be found by using the method of the polar and funicular polygon (see Fig. 367).

Divide the section into strips, and draw the lines passing through the centres of the strips; let the lengths of these centre lines $a_1 a_2 \ldots a_8$ represent the areas of the strips. Construct a polar polygon from these centre lines, let A = the area of the section, making the pole a distance $\frac{A}{2}$ from PQ. Construct a funicular polygon *mn* . . . . *tz*. The intersection of the reciprocals PO, QO at Z determines a point in the line passing through the CG.

Then to determine the moment of inertia produce *sr* and *ts* to cut CG in *b* and *d*, then

$$\frac{bd}{x} = \frac{a_6}{\frac{A}{2}}$$

$$bd = \frac{2\,a_6 \times x}{A}$$

multiply both sides by x

$$bdx = \frac{2\,a_6 \times x^2}{A}$$

Let the partial area $bds = \delta a_1$

$$\text{then } \delta a_1 = \frac{bd \times x}{2} = \frac{a_6 \times x^2}{A}.$$

The sum of all the partial areas

$$\Sigma\, \delta a_1 = A_1$$

$$\therefore A_1 = \frac{\Sigma\, a_6 x^2}{A}$$

$$\text{and } A_1 A = \Sigma\, a\, x^2 = I.$$

That is, the area of the section or area multiplied by the area of the funicular polygon equals the moment of inertia about a line passing through the centre of gravity.

The Radius of Gyration r of an area about a given axis is that distance whose square is the mean of the squares of all the distances of the indefinitely small particles of the area from the axis.

$$\text{Then } A_1 \times A = I$$

$$A_1 = \frac{I}{A} = r^2.$$

That is, the area of the funicular polygon equals the square on the radius of gyration.

To determine the Moment of Inertia about any other axis, say the line XX_1. Produce the extreme links of the funicular polygon till they meet in z. Produce Zm and tz till they meet the desired axis XX_1. Call the intercept on XX x, and the perpendicular from x to z, y.

Then the moment of inertia about any axis equals the product of

$$A \times \text{the area of the funicular} \times m \ldots tx_1.$$

Let the area of the original polygon $mn \ldots tz = A_1$ and the area of the triangle $XXz = A_2$.

Then the moment of inertia about XX

$$I_{xx} = A(A_1 + A_2)$$

A and A_1 are known, to determine A_2 from the diagram

$$\frac{x_1}{y} = \frac{A}{\frac{A}{2}} = 2$$

$$x_1 = 2y.$$

$\therefore$ the area of the triangle XXz

$$= y \times y = y^2.$$

$$\text{Then } I_{xx} = AA_1 + A y^2$$

$$I_{xx} = I_{cg} + A y^2.$$

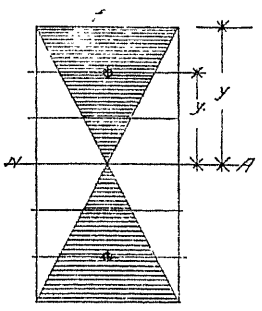
Resistance Areas.—The resistance area of any section. The stress in any layer of a beam section parallel to the neutral axis varies directly as its distance y from the neutral axis; and if the width of the layers be reduced *till the intensity of stress is the same throughout*, the resulting figure is known as the Resistance Area.

Let a be the area and y distance of any layer from the NA and a_1 the area and y_1 the distance of any other layer, and f and f_1 the stresses on any layer.

$$\text{Then } \frac{a_1}{a} = \frac{y_1}{y} = \frac{f_1}{f}$$

$$a_1 = \frac{y_1 a}{y}.$$

The Resistance Area of any figure may be obtained by drawing any two axes XX and YY through the extreme



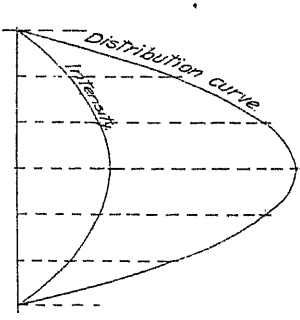
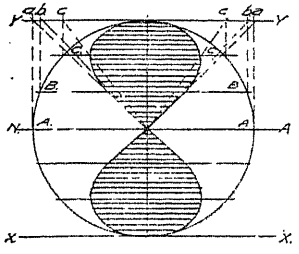


Fig. 368.



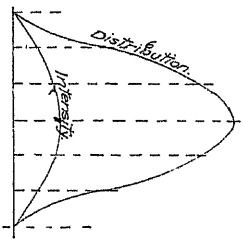
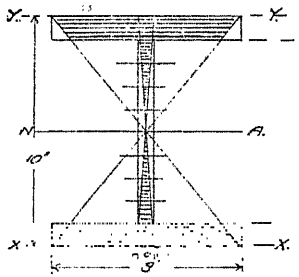


Fig. 369.



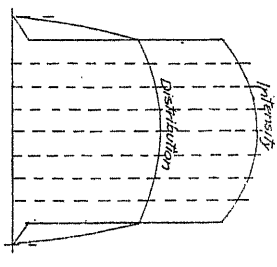
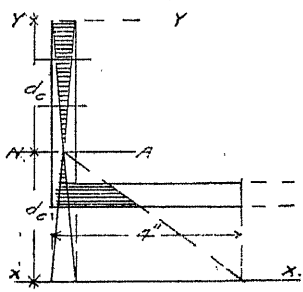


Fig. 370.



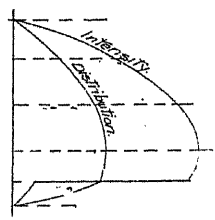


Fig. 371.

Figs. 368—371.

layers of the section and parallel to the neutral axis (Figs. 368 to 371). Divide the section up into any convenient number of layers, project the extremities of these layers on to the axes XX or YY on their respective sides of the NA. Then draw lines from these points radiating to the pole in the CG of the section. The points of intersection between the radiating lines and their respective horizontals are points in the equivalent figure, which latter is known as the resistance area. Figs. 368 to 371 give the methods of determining the resistance areas for a rectangle circle, I and L sections. The resistance areas of other sections may be obtained in a similar manner.

Arches.—Arches may be considered as bent beams. If an upward curve be given to a hitherto straight beam, the tension on the lower fibres will be reduced, and if the curve be increased till the ordinates to the curve from the chord line are coincident with the bending moment at all parts, the tension will be eliminated and the whole of the beam will be subjected to a compressional stress. This is the ideal condition for a masonry arch, as the mortar is not considered reliable to resist tensional stresses. If the load is uniformly distributed the bending moment curve will be a parabola, but any change in the position of the loads would modify this curve and tension in some part of the arch rib would result from the redistribution of the load.

In all arches there is an outward horizontal thrust on the abutments which must be known in order that the stresses in the other parts of the arch may be determined. In all arches with the exception of the three-pinned and the parabolic arch the horizontal thrust is statically indeterminate. In other forms the elasticity of the material and temperature stresses have to be considered. In all types of arches the bending moment at any part will be the algebraic sum of the bending moment calculated as for a straight beam, and the moment of the horizontal thrust into y the height of the point on the arch axes under consideration.

That is $$M = M_1 + Hy$$

where M_1 is the moment calculated as for a straight beam,

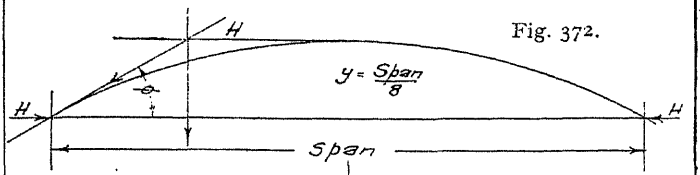


Fig. 372.

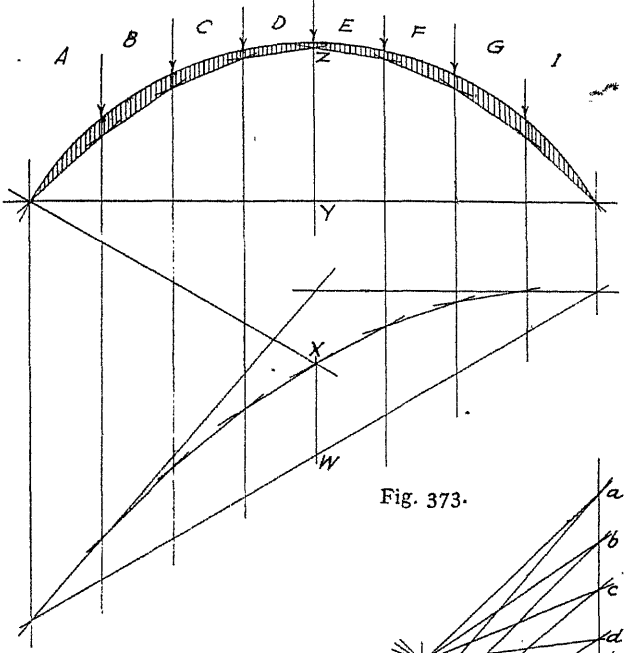


Fig. 373.

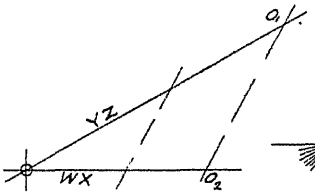


Fig. 374.

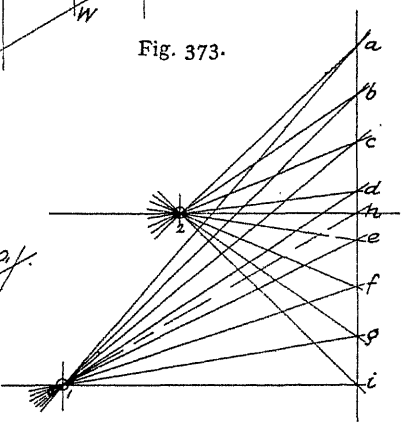


Fig. 375.

Figs. 372—375.

H is the horizontal thrust and y the height from the springing to the arch axes.

In the case of the parabolic arch under a uniformly distributed load the line of pressure coincides with the centre line of the arch ring; therefore there will be no bending moment, or the moment of the horizontal thrust equals the bending moment at any point, that is

$$Hy = \frac{wl^2}{8}$$

and

$$H = \frac{wl^2}{8y}$$

and the thrust in the ring at any point will equal

$$T = H \sec \theta. \quad \text{(See Fig. 372.)}$$

Arches, especially masonry arches, are usually built to some segment of a circle, in which case it is advisable to select a segment that will approximately coincide with the curve of a parabola. This will be found to be when the rise of the arch equals one-eighth of the span. In all other cases of segmental arches under a uniformly distributed load there will be a bending moment which will be equal to the difference in the ordinates of the arch ring and the funicular on the same base (see Figs. 372 to 375).

That is $M = M_1 + Hy$

And if M_1 be positive then Hy will be negative and

$$M = M_1 + (-Hy)$$

Three-pinned Arch.—The three-pinned arch is usually employed with steel or ferro arches of long span. There are three pins or pivots one at each springing point and one at the centre. With this arrangement the line of stress must pass through the pivots on the arch rib at which points the B.M. is zero (see Figs. 376 to 379).

Then $M = M_c + Hy_c$

And M at the centre = Zero

$$M = 0 = M_c + Hy_c$$

and

$$H = -\frac{M_c}{y_c}$$

at any other point

$$M = M_1 + Hy = M_1 - \frac{M_c y}{y_c}$$

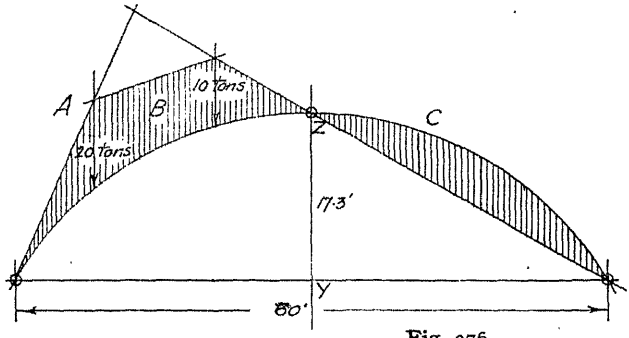


Fig. 376.

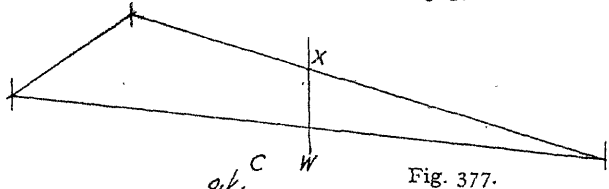


Fig. 377.

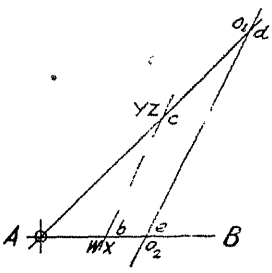


Fig. 378.

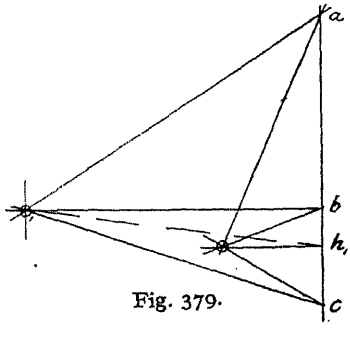


Fig. 379.

Figs. 376—379.

or the bending moment equals the difference between the calculated BM as for a straight beam and the arch rib.

To set out a graphical example, take an arch 60 feet span with a rise of 17·3 feet with loads of 20 and 10 tons at 8 feet and 20 feet respectively from the left-hand support. Scale for the space diagram : 10 feet = 1 inch. Draw the vector polygon. Scale : 10 tons = 1 inch. Take a pole O_1 anywhere, say, 3 inches measured at right angles from the vector polygon. Draw the polar lines, and with parallels from these drawn across the spaces A, B, C and D on the space diagram, draw the funicular polygon and the reciprocal to the closing line on the polar polygon. In order that the BM may be readily measured, it is necessary to draw a second funicular polygon with its extremities passing through the two springing pins of the arch. To do this, a new pole is required. On the space diagram draw a perpendicular through the centre pin of the arch and passing through the first funicular. Then figure the intercept passing through the funicular WX and that passing through the arch ring YZ.

Then $$WX : YZ :: O_1 : O_2$$

and $$O_2 = \frac{YZ \times O_1}{WX} = \frac{0{\cdot}6 \times 3}{1{\cdot}75} = 1{\cdot}028$$

Or to obtain this graphically, see Fig. 378. Draw two lines AB, AC making any angle with each other. Along AB mark the length WX, giving A*b* and on AC mark the length YZ giving AC. Join *bc*. Then from A along AC mark off A*d* the length o_1h_1 ; from *d* draw a parallel to *cb*, cutting AB in *e*. Then A*e* will be the fourth proportional to WX, YZ and o_1h_1 and will be the length of the new pole o_2h_2. From *d* on the vector polygon draw a parallel to o_1h_1 and mark off the length o_2h_2. From o_2 draw a new set of polar lines to *a*, *b* and *c* on the vector polygon, and commencing at P draw parallels to the new polar lines. These will pass through Z on the centre pin of the arch rib. Then the intercepts shown between the linear arch and the arch rib at any point measured with the bending moment scale will equal the bending moment in the rib. The horizontal thrust is equal to o_2h_2.

The bending moment scale is the product of the linear scale × force scale × o_2h_2.

BM scale = 10 × 10 × 1·05″ = 105 tons feet = 1″

The principles governing the distribution of the loads in masonry arches can only be considered approximate, as the amount of the load due to the spandril wall above, and its application cannot be accurately determined, but if the whole of the spandril between vertical lines drawn from the springing point on the extrados be considered to act vertically, then the error will be on the safe side. The masonry arch, consisting of a number of separate blocks, unlike the steel rib, has no capacity for resisting tension. The arch must therefore be made of such a form, or the load must be so distributed that the line of thrust falls within the middle third of the arch ring. The whole of the section of the rib will then be subjected to a compressional stress.

In large arches the live load is generally a negligible quantity compared with the weight of the structure itself. The effect, however, of the live load in different positions on the arch should be tried to determine whether any modifications are required in the form or the depth of the arch. Take the case of an arch of 60 feet span with a rise of 7 ft. 6 in. Radius = 63 ft. 9 in. Supports a spandril, as shown in Fig. 380, 5 ft. 6 in. in depth above the centre line of the arch at the crown. Then the depth of the arch from Rankine's formula =

$$\begin{aligned} D &= \sqrt{\cdot 12 \times \text{radius at crown}} \\ &= \sqrt{\cdot 12 \times 63' 9''} \\ &= 2{\cdot}65, \text{ say } 2' 6''. \end{aligned}$$

Then set out the arch to the above dimensions measured along the centre line of the arch. Divide the arch into six equal sections, A, B, C, D, E. The weight of these may, without any sensible error, be assumed to act along their centre lines, and to be in proportion to their length. Set out the vector polygon, making the total length equal to 10·2 tons. Take any pole o_1 and draw the polar lines, and from this the funicular polygon. The intersection of the lines o_1a

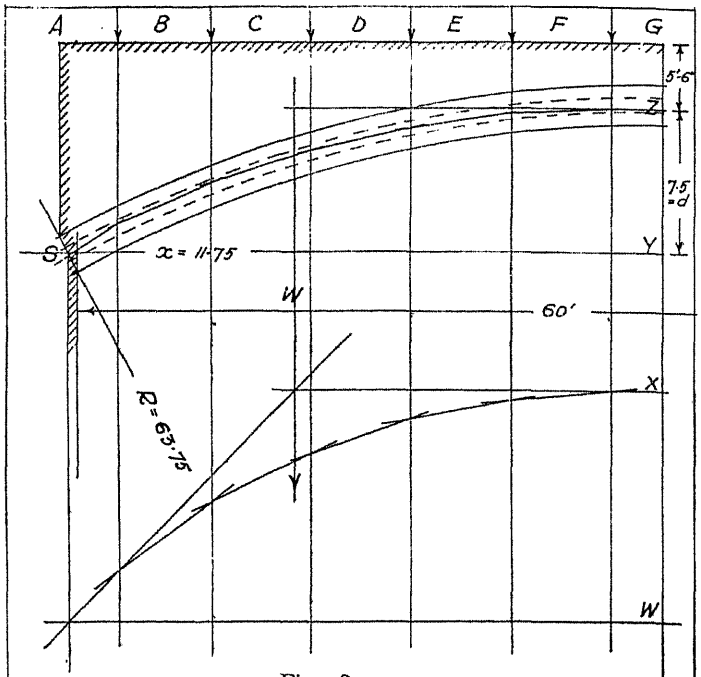


Fig. 380.

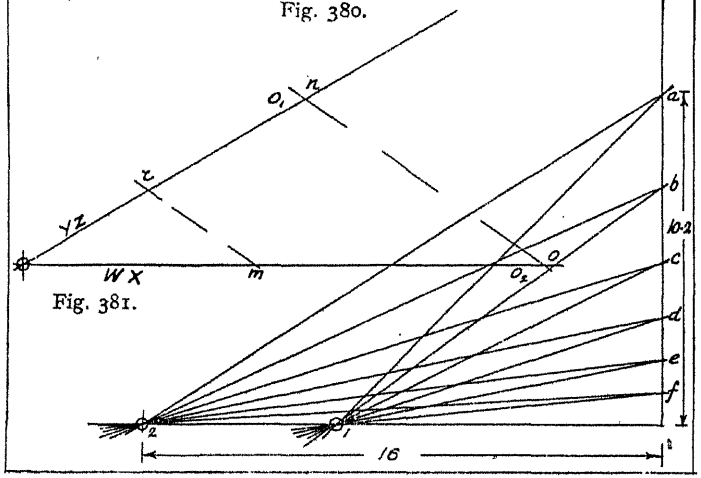


Fig. 381.

Fig. 382.

Figs. 380—382.

and o_1g will give a point in a vertical line passing through the centroid of the spandril.

Then
$$7{\cdot}5\ \mathrm{H} = \mathrm{W}x$$
$$\mathrm{H} = \frac{10{\cdot}2 \times 11{\cdot}75}{7{\cdot}5}$$
$$\mathrm{H} = 16 = o_2g$$

Set out the new pole o_2g and draw the new polar lines and the second funicular or line of stress starting from the point S; it will be found to lie within the middle third of the arch rib. That the line of stress is not quite coincident with the arch rib is due to the weights of the sections not being quite uniform. They vary from a maximum at the haunch to a minimum at the crown. The new length of o_2g could have been found by setting out two lines at any angle and measuring the length of WX from the first funicular on one line and the length of YZ from the space diagram. Join these lengths, and then on the line YZ set out the length of o_1; from the point draw *no* parallel to *lm*. This will give the length o_2g (see Figs. 380 to 382).

EXAMPLE I.—A buttress of masonry weighing 140 lbs. per cubic foot, having the form of accompanying diagram (Fig. 383), and of a uniform width of 5 feet, with a pyramidal top, has to sustain the two thrusts shown. Determine if the buttress is likely to overturn about XX or ZZ.

To obtain the Weight.—The figure may be considered to be made of six blocks. First determine the weight of each block, and find the vertical line in which the mass may be supposed to be collected, and acting through the force of gravity.

In this example it is only necessary to determine the weight of each block, starting with top, and to be able to obtain the centres of gravity of prisms, pyramids, and blocks of quadrilateral section, and by means of the method shown, obtain the centre of parallel forces, and find the resultant pressure of No. 1 and No. 2 blocks, then the resultant of three blocks, then the resultant which is shown as DF in the diagram of the four blocks and the upper thrust R.

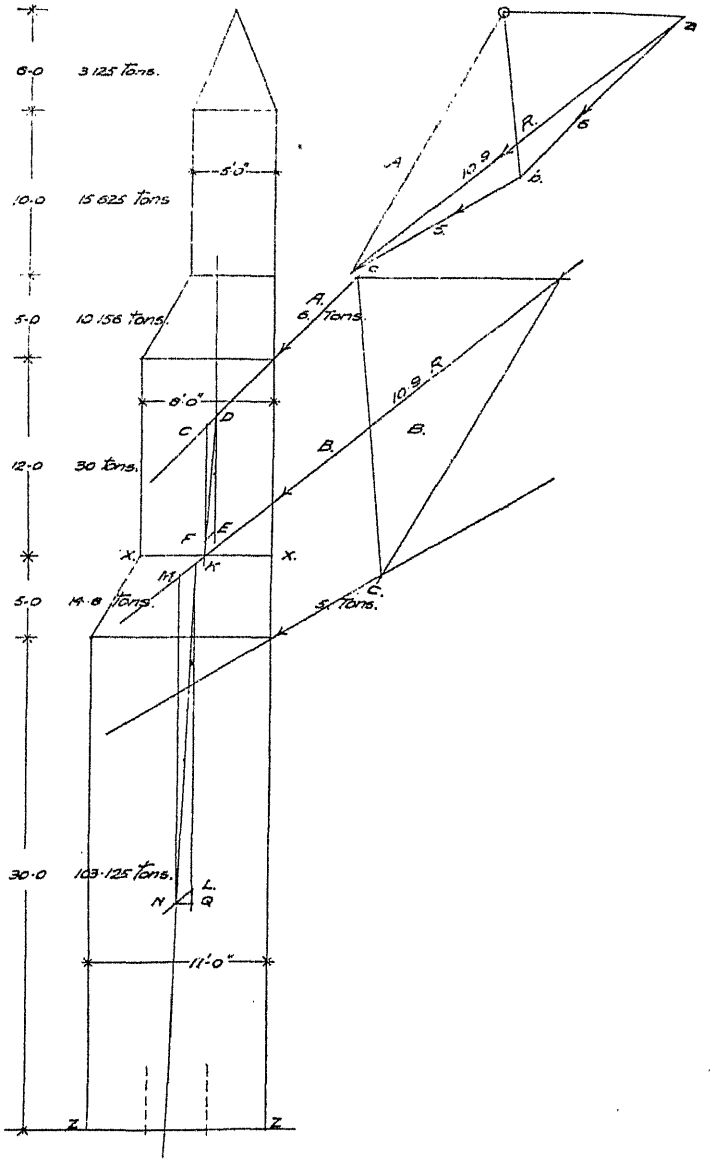

6-0
3.125 Tons.
5'0"
10-0
15.625 Tons
5-0
10.156 Tons.
A.
6. Tons.
8'0"
C
D
10.9 R.
B.
B.
12-0
30 Tons.
F
E
X.
X.
M.
K
5-0
14.8 Tons.
C.
5. Tons.
30-0
103.125 Tons.
L.
N
Q
11'0"
Z
Z
a
R.
6
A
10.9
b.
5.
c

Fig. 383.

Taking the line AB, representing the upper thrust, produce it, and at the point of intersection with the vertical line representing the pathway of the aggregate pressure of the mass of the four blocks, draw a parallelogram, the side CD being set out to scale with measure, 6 units, and DE 58·9 units; the line DF will give the resultant pressure to scale and the direction.

The buttress at this joint will be safe (1) from overturning if the resultant falls within the base XX, and (2) if it intersects within the middle third will be safe from tension at that section, provided that twice the normal pressure does not exceed the compressional strength of the material, and (3) will be safe from sliding if the angle between the resultant DF and FC does not exceed the angle of which four-fifths of the coefficient of friction of the material is the tangent. This is fully described in the chapter on Brickwork.

In a like manner, the section at ZZ may be tested by obtaining: first, the sum of the total vertical loads and the centre of pressure at which it may be supposed to act; second, the resultant R of the two thrusts AB and BC.

To determine the direction and the point of application of the two thrusts, draw a reciprocal diagram and the closing line *ac* of the two forces AB and BC. Select a pole and join *oa*, *ob*, *oc*. *ac* will be the resultant. Its point of application may be found by drawing a link polygon. Draw a parallel to *ob* across the space B and from its point of intersection with AB draw a parallel to *oa*; also from its intersection with BC draw a parallel to *oc*. The intersection of these two parallels will give a point in the pathway of the resultant force. From this point draw a parallel to *ac*, and produce till it cuts the vertical drawn through the common line that passes through the CG of the whole mass. Let these be represented by KL and KM, then KN will be the resultant pressure. The total normal compression is KQ, which is the value of the vertical component of the thrust KN, and it is well to note that the buttress is safe from overturning and compression at the bed joint ZZ.

Notation.—In the application of the foregoing principles to actual examples it will facilitate the working if the following, known as Bow's, method of notation is employed, *i.e.*, the spaces between all forces or bars are lettered or numbered, so that any force or bar is known by the letters or numbers on each side of it, and the stresses on the force polygon are known by the letters or numbers at the extremities.

Limits of Application.—To be possible to apply the laws of graphic statics for all conditions of loading and fixing, all frame polygons must (1) be built up of a number of bars forming a triangulated system, and if S = number of bars, p = number of angular points, then $S = 2p-3$, which is the relation that sides and angular points of all triangles have to each other; (2) three forces at least, no two being in the same straight line, must radiate from each of its nuclei, or the figure must be considered to be solid and unalterable. Compliance with the above conditions cannot always be obtained, and in order to apply the laws of graphic statics to such structures advantage must be taken of various subterfuges for a solution. The following common cases are dealt with: (1) Frames having an insufficient number of members, as in the queen post truss (Fig. 385) and in the French truss with kneebraces (Fig. 386); (2) frames with redundant members, as in the example of a lattice girder (Fig. 387); (3) frames in which the stresses in more than two bars meeting at a point are unknown as in the Belgian truss (Fig. 388).

Loading of Frames.—Frames may be subjected to dead loads acting in a vertical direction or inclined loads caused by wind or other side thrusts, or by a combination of both. The first two cases, as a rule, present no difficulty, as the reactions can be easily determined. The third case can usually be solved with the aid of polar and funicular diagrams, but some cases of this class are more easily solved by determining the inclined and vertical stresses separately and by adding the results to obtain the final result. The loads should in all cases be applied at the nodal points of a truss and not along the bars between. In the

latter case the members would be subject to transverse stress, instead of, as in the former case, to direct tension or compression.

Many steel frames above 40 feet in span are provided with expansion rollers at one end and are fixed at the other, or both ends are fixed. The method of support affects the direction of the reactions. The reaction under rollers is of necessity vertical and the fixed reaction is inclined. In the second case both reactions are equally inclined. In both cases the magnitude of the reactions can be determined by the funicular and polar polygons. The dead loads consist of the weight of the covering, rafters, purlins, trusses and ceilings. These can be approximately worked out from the table of the weights of roofing materials appended. The total weight being divided by the superficial area of the covering gives the load per square foot, which is a convenient form. The actual load can only be determined when the final dimensions of the trusses are known. The wind is assumed to act on one side of the roof in all the examples shown. Recent experiments at the National Physical Laboratory tend to show that the pressure of the wind on the roof surface on the windward side is not uniform, and that on the leeward slope there is a suction force, due to the reduced air pressure caused by the wind passing over the ridge; the effect of this seems to be to cause a reduction in the pressures, or even a reversal of the stresses, on some of the bars of the roof. To obviate any danger due to a possible reversal of stresses all bars should be made of a section that would be efficient under compression, and in the case of light roofs the frames should be bolted down. The maximum stresses in the bars obtained by ignoring suction on the leeward side would appear to be on the safe side. The horizontal values of wind pressure are given in the article on the stability of walls, and it is sufficient to note that for ordinary practice the wind may be considered to exert a force not exceeding 30 lbs. per square foot on a surface perpendicular to its direction and to obtain the normal pressure per square foot (Pn) on slope of roof, this must be multiplied by a factor which varies for each inclination. This table is the result of numerous experiments.

TABLE SHOWING THE VALUES of P*n* when P = 1 lb. per Square Foot. (*From Hurst.*)

Inclination of Surface — *i*.	P*n* Normal to Surface.	Inclination of Surface — *i*.	P*n* Normal to Surface.
Degrees.	lbs.	Degrees.	lbs.
5	·130	50	·952
10	·240	55	·968
15	·349	60	·980
20	·456	65	·987
25	·549	70	·993
27	·603	75	·996
30	·662	80	·998
35	·754	85	·999
40	·834	90	1·000
45	·900		

TABLE GIVING WEIGHT OF ROOF MATERIALS.

(*From Hurst.*)

	lbs. per foot super.
Common rafters and purlins (wood trusses)	7
,, ,, ,, (iron trusses)	6
One-inch deal boarding...	3·33
Battens 3″ × ¾″ laid 8½ inch gauge	1·32
Asphalted felt	0·5
Zinc, laid with rolls, 12 inch gauge	1·5
,, ,, ,, 14 inch gauge	1·7
,, ,, ,, 16 inch gauge	1·9
Lead and copper, according to weight per foot, adding one tenth for seams and laps.	
Corrugated iron, 20 BW gauge, with 5 inch corrugations (measured overall)	1·90
,, 18 ,, ,, ,, ,, ,,	2·60
Slates, 3 inch lap, all sizes, except Rags or Queens, including nails	9·0
Rags and Queens, including nails	12·0
Tiles, plain, 10½″ × 6″ × ¾″ including mortar for pointing, 4 inch gauge	16·0
Ceiling, including joists, 10 feet bearing and lath and plaster	12·0
Snow, according to climate	3 to 10·0

Application.—Difficulties in the solution of statical problems will be cleared away if the student observes the following order of operations :—

1. Draw the proposed frame polygon, and note, if an open frame, if it satisfies the condition $S = 2p - 3$.

2. Position of forces, including reactions (exact or approximate).

3. Notation.

4. Draw as much as possible of the force polygon, leaving only two forces of which the two directions, or the two magnitudes, or the two magnitudes and one direction are to be determined, the algebraical sum of these effects with one direction only being known.

5. Draw polar point, and all polar lines but one.

6. Draw the polar polygon.

7. Determine reactions by drawing the reciprocal polar line of the closing line of polar polygon to intersect the reaction, the direction of which is known. The line closing the force polygon will give the magnitude and direction of the unknown reaction.

8. Draw stress diagram.

EXAMPLE.—Determine the stresses on the roof shown on Fig. 384; span, 50 feet; pitch, 30 degrees; trusses, 12 feet, centre to centre; one end fixed, one end on rollers; dead load, 30 lbs. per square foot; wind load, normal to slope, 20 lbs. per square foot; ceiling load, 12 lbs. per square foot.

Then dead load	$=$	$\frac{29 \times 2 \times 12 \times 30}{2240}$	$= 9{\cdot}35$ tons.
Load on central points	$=$	$\frac{9{\cdot}35}{8}$	$= 1{\cdot}17$ tons.
Load on end points			$= {\cdot}585$ ton.
Wind load .	$=$	$29 \times 12 \times 20$	$= 3{\cdot}1$ tons.
Load on central points	$=$	$\frac{3{\cdot}1}{4}$	$=$ say ${\cdot}8$ ton.
Load on end points			$= {\cdot}4$ ton.
Ceiling load	$=$	$50 \times 12 \times 12$	$= 3{\cdot}2$ tons.
Load on central points	$=$	$\frac{3{\cdot}2}{8}$	$= {\cdot}4$ ton.
Load on end points			$= {\cdot}2$ ton.

The latter are taken directly by the walls and need not further be taken into account.

Then, in the order of operations recommended, draw the frame polygon. (1) Indicate on the figure the forces, combining the wind and dead loads on the windward side of the roof. (2) Apply the method of notation to identify

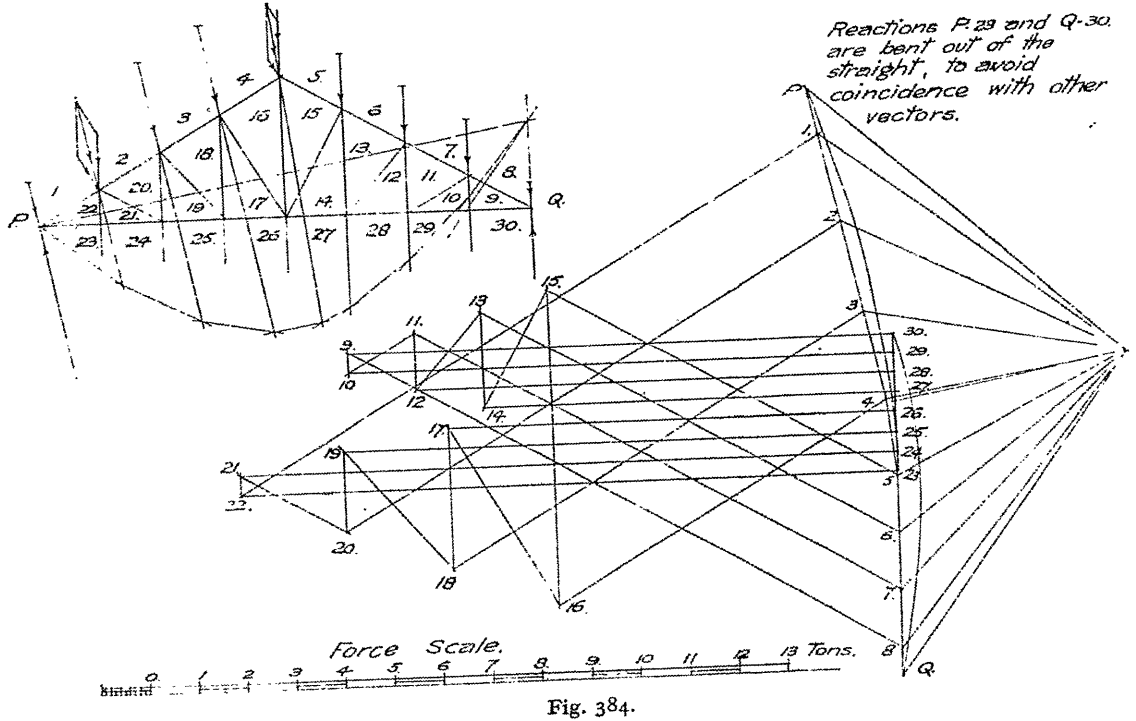

Reactions P.29 and Q-30. are bent out of the straight, to avoid coincidence with other vectors.
Force Scale,
0 1 2 3 4 5 6 7 8 9 10 11 12 13 Tons.

Fig. 384.

the forces. (3) Draw the force polygon from P to Q. Draw the reactions as though there was no ceiling load. The reaction under roller at Q will be vertical. To determine its magnitude, draw a polar and link polygon from the point O; on the polar polygon draw a parallel to the closing line on the link polygon. If there had been no ceiling loads the line drawn from this intersection to the point P would have given the magnitude and direction of the reaction at P. The addition of the ceiling loads will obviously increase the reactions and cause that at P to assume a more vertical direction. From the point of intersection of the vertical reaction and the parallel to the closing line of the link polygon set out a vertical line and mark off on each side of the point on half of the seven intermediate ceiling loads 23—24, 24—25, 25—26, 26—27, 27—28, 28—29, 29—30, marking off on this line the magnitudes of each of the loads. Then join P to the lowest point 23 and Q to the highest point 30; this gives the magnitude and direction of the two reactions. These two lines, representing the reactions, have been bent for the sake of clearness to avoid coincidence with the other vectors. The whole diagram may now be completed by drawing the reciprocals of all the bars on the frame diagram. The loads on each bar can be measured with the force scale and the values tabulated for the purpose of computing the sectional areas.

EXAMPLE.—Given a queen post truss 41·5 feet effective span. Pitch, ¼th. Trusses, 15 feet centres. Dead load, 25 lbs. per square foot. Wind to normal slope, 20 lbs. per square foot. Determine the loads on the members (see Fig. 385).

Dead load	$= \frac{23{\cdot}2 \times 15 \times 2 \times 25}{2240}$	$= 7{\cdot}75$ tons.
Dead load on each node	$= \frac{7{\cdot}75}{6}$	$= 1{\cdot}29$ tons.
Dead load on end node		$= 0{\cdot}65$ ton.
Wind load	$\frac{1}{2240}(23{\cdot}2 \times 15 \times 20)$	$= 3{\cdot}11$ tons.
Wind load on central nodes	$= \frac{3{\cdot}11}{3}$	$= 1{\cdot}04$ ton.
Wind load on end nodes		$= 0{\cdot}52$ ton.

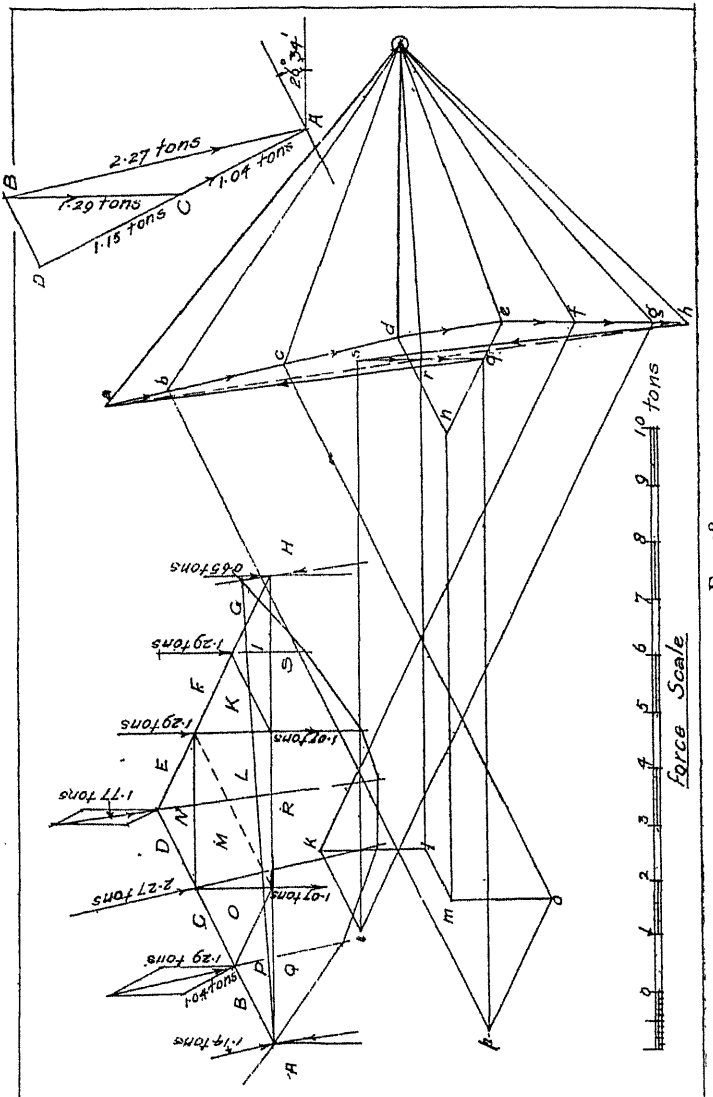

2·27 tons
1·29 tons
1·04 tons
1·15 tons
Force Scale
10 tons
0·65 tons
1·29 tons
1·07 tons
1·77 tons
2·27 tons
1·29 tons
1·04 tons
1·14 tons

Fig. 385.

Resolving the wind and dead loads on the windward side of roof for the resultants at the nodes. This may be done by the parallelogram of forces on the diagram or from the formula—

Inclination of rafter 26° 34′.

$$AB = \sqrt{AC^2 + CB^2 + 2AC \cdot CB \cos 26° 34'}$$
$$= \sqrt{1{\cdot}04^2 + 1{\cdot}29^2 + 2 \times 1{\cdot}04 \times 1{\cdot}29 \times 0{\cdot}894}$$

AB = 2·27 tons on central nodes.
and 1·14 tons on lowest point.

$$DC = BC \cos BCD = 1{\cdot}29 \cos 26° 24'$$
$$= 1{\cdot}15$$

The inclination of the resultant to the rafter θ

$$\sin \theta = \frac{AC + DC}{AB} = \frac{1{\cdot}04 + 1{\cdot}15}{2{\cdot}27} = 0{\cdot}966$$
$$\theta = 75°$$

In a like manner the resultant of the wind and dead loads at the apex of the roof = 1·77 tons acting at an angle of 70° 34′

Draw these external loads on the frame diagram, and the reciprocals as the force polygon. To determine the reactions draw *a* from *h* to *a*, which gives the direction and magnitude of both reactions.

To obtain their individual values take a pole at any point and draw a polar diagram, and commencing at the point A on the frame diagram, draw a parallel to *bo* on the polar polygon, and produce the force BC to intersect. Continue this till the point passing through H on the funicular polygon is reached at the point X. Join X to A to close the funicular, and draw a parallel from O on the polar polygon to meet the line *ha* at the point *r* on the force polygon.

If a ceiling is suspended from the roof, and taking the weight of ceiling joists and plaster as 12 lbs. per foot super, the weight of the ceiling on one truss would be

$$W_c = \frac{40 \times 15 \times 12}{2240} = 3{\cdot}22 \text{ tons.}$$

The load on the two central points will be

$$\frac{3{\cdot}22}{3} = 1{\cdot}07 \text{ tons.}$$

The weight at the extremities will be taken directly by the wall and need not be considered.

Draw a vertical through the point r on the line ah on the force polygon and measure from r above and below 1·07 tons. Then draw a line from h to the upper point s and from a to the lower point q; this will give a modified amount and direction to the two reactions. Having obtained the limits and magnitudes of all the external forces acting on the frame the diagram can be completed by drawing parallels to the bars in the frame. As this truss is incomplete, the diagram will not close unless the dotted bar ML is inserted in the frame. If this bar is omitted in the actual truss the tie beam will be subjected to a bending stress. Where these bars are inserted it is usual to insert two for the sake of symmetry.

French Roof Truss with Knee Braces.—The essential difference between this and the ordinary types of trusses, is that the effect of the wind on the wall surface must be taken into account as well as the roof. The structure becomes a braced portal in which the truss forms the top member.

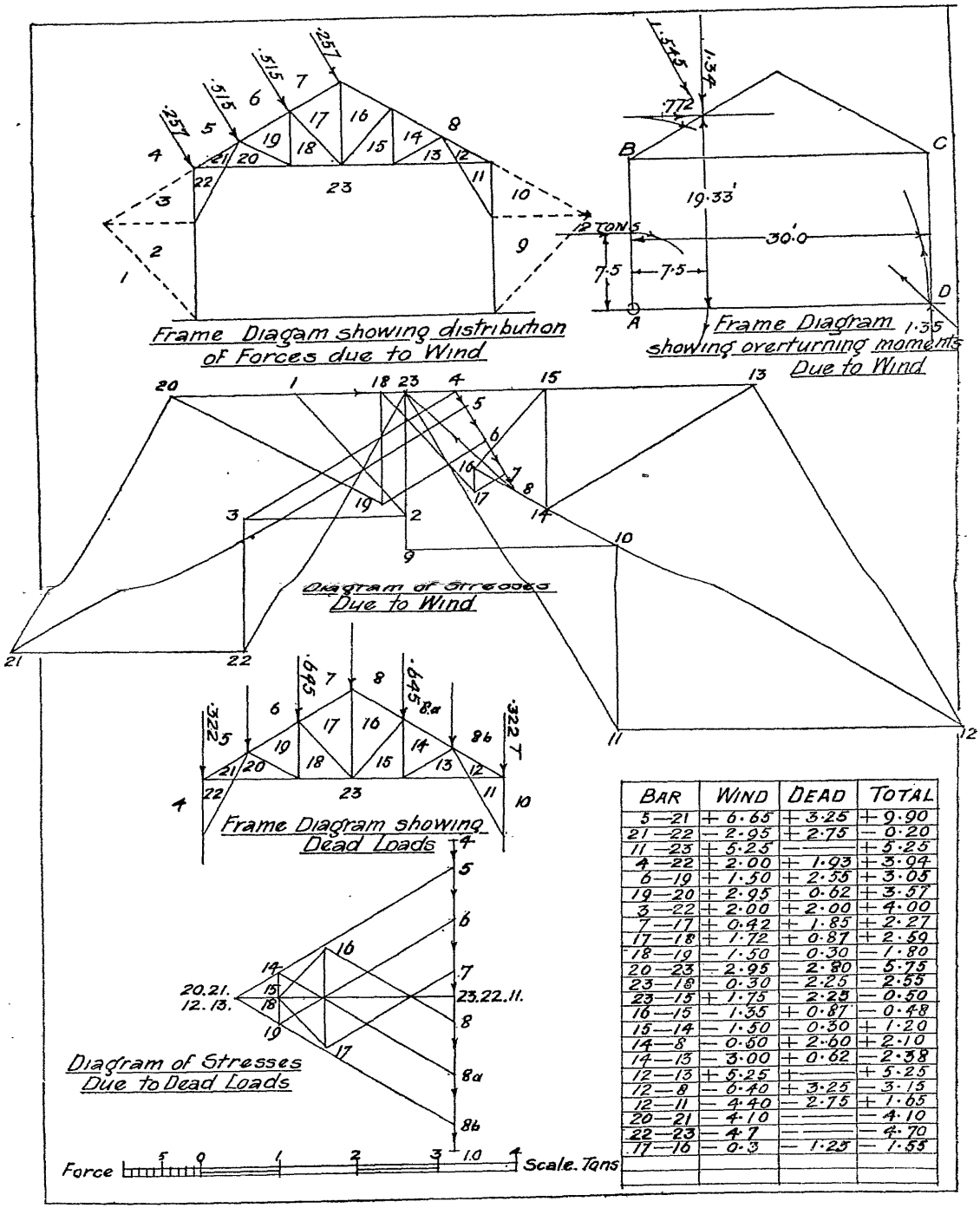
For simplicity it is best to consider the effect of the wind and the dead loads separately and to add the effects to obtain the total stress on the members.

EXAMPLE.—Determine the stresses on the bars of a French roof truss (Fig. 386). Span, 30 feet. Pitch, 30 degrees. Height, 15 feet to eaves. Trusses, 10 feet centres. Wind, 30 lbs. per square foot horizontally, 20 lbs. normal to the slope. Dead load, 25 lbs. per square foot.

Then wind load at each central point $= \frac{10 \times 17{\cdot}3 \times 20}{3 \times 2240} = {\cdot}515$ ton

On each end point $= {\cdot}257$ ton

Resultant wind load on slope $= \frac{10 \times 17{\cdot}3 \times 20}{2240} = 1{\cdot}545$ tons



BAR	WIND	DEAD	TOTAL
5—21	+ 6·65	+ 3·25	+ 9·90
21—22	— 2·95	+ 2·75	— 0·20
11—23	+ 5·25	—	+ 5·25
4—22	+ 2·00	+ 1·93	+ 3·94
6—19	+ 1·50	+ 2·55	+ 3·05
19—20	+ 2·95	+ 0·62	+ 3·57
3—22	+ 2·00	+ 2·00	+ 4·00
7—17	+ 0·42	+ 1·85	+ 2·27
17—18	+ 1·72	+ 0·87	+ 2·59
18—19	— 1·50	— 0·30	— 1·80
20—23	— 2·95	— 2·80	— 5·75
23—18	— 0·30	— 2·25	— 2·55
23—15	+ 1·75	— 2·25	— 0·50
16—15	— 1·55	+ 0·87	— 0·48
15—14	— 1·50	— 0·30	+ 1·20
14—8	— 0·50	+ 2·60	+ 2·10
14—13	— 3·00	+ 0·62	— 2·38
12—13	+ 5·25	—	+ 5·25
12—8	— 0·40	+ 3·25	— 3·15
12—11	— 4·40	— 2·75	+ 1·65
20—21	— 4·10	—	— 4·10
22—23	— 4·7	—	— 4·70
17—16	— 0·3	— 1·25	— 1·55

FIG. 386.

[*Between pages* 502 *and* 503.

Vertical component of wind on slope $= \frac{\sqrt{3} \times 1{\cdot}545}{2} = 1{\cdot}34$ tons

Horizontal component of wind on slope $= \frac{1{\cdot}545}{2} = {\cdot}772$ ton

Total wind on wall surface $= \frac{10 \times 15 \times 30}{2240} = 2$ tons

To determine the upward reaction of the external forces at D take moments about A.

D × lev. = (wind on wall × lev.) + (vertical comp. of wind on slope × lev.) + (horizontal comp. of wind on slope × lev.)

$$D \times 30 = 2 \times 7{\cdot}5 + 1{\cdot}34 \times 7{\cdot}5 + {\cdot}772 \times 19{\cdot}33$$
$$= 15 + 10{\cdot}05 + 14{\cdot}92$$
$$= \frac{40}{30} = 1{\cdot}35 \text{ tons.}$$

Draw the diagram for the wind load only.

The horizontal components of the reactions at A and D are assumed to be equal. Therefore half the total horizontal forces

$$= \frac{2 + 0{\cdot}772}{2} = 1{\cdot}39 \text{ tons}$$

The reciprocal diagram of the external forces can now be drawn.

Set 1·39 tons off from 1 to give the point 23. Join this to 8, 8 to 23, 23 to 1. This completes the frame diagram.

As the frame is incomplete and as the uprights are assumed free to rotate about A and D the members shown dotted in the frame diagram must be added. The diagram may now be completed by drawing parallels to the bars in the frame polygon.

Dead load at each central point on sloping surface

$$= \frac{2 \times 10 \times 17{\cdot}3 \times 25}{2240 \times 6} = {\cdot}645 \text{ ton.}$$

Dead load on end points = ·3225 ton.

As the dead load is uniformly spread over the surface of the roof and the points of support are symmetrical, the

reactions are equal. Proceed to plot the force polygon, on which point 23 will be central. The remainder of the diagram can be drawn without further difficulty. It will be seen that no stresses are taken by the knee braces from the dead load. Care must be taken to ascertain the sense of the stress in the bars, indicating the compressional members in the table by a plus sign and the tensional members by a minus sign. The algebraic sum of the loads due to the wind and the dead load should then be entered in the total column. The dotted members being non-existent the upright supports will be subject to bending, the greatest moment being at the foot of the knee brace.

M at foot of knee brace = the horizontal component of the reaction at A or D × distance from A or D to the foot of the knee brace.

$$= \frac{(2 + 0{\cdot}772)10}{2} = 13{\cdot}86 \text{ tons feet.}$$

Lattice Girder.—EXAMPLE.—Determine the stresses on the members of a lattice girder covering a span of 60 feet and the depth of which is 10 feet, and having lattice bars, as shown in Fig. 387 at an angle of 45 degrees.

Application.—The order of the eight operations should be followed and worked.

Where frames have redundant members, there is often a difficulty in making the stress diagram close. The closing process may frequently be done by making certain assumptions (see Fig. 387). This example shows a lattice girder with equal loads at uniform intervals. In this case it is evident that the thrust in the members P—19 and Q—7 must be known before the value of the adjoining members can be obtained. Upon inspection it is evident that the truss is a combination of two systems of triangulation and that P—19 and Q—7 will be the reactions to the loads P—1, 2—3, 4—5 and 6—Q, and as the loads are symmetrical the reactions will be equal and may be set down on the force diagram. The points 19 and 7 being known, the whole diagram can be completed. The stresses on the members may be determined by working out the two systems separ-

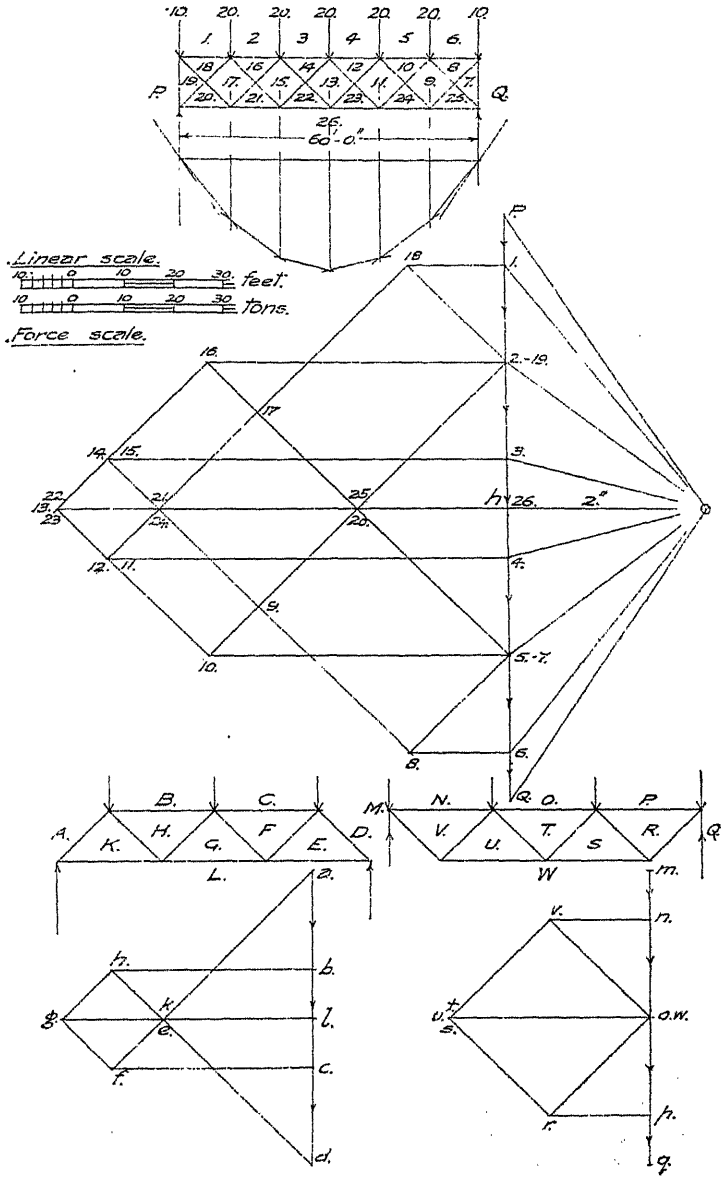

10. 20. 20. 20. 20. 20. 10.
1. 2 3 4 5 6.
P. Q.
26.
60'-0"
Linear scale.
feet.
tons.
Force scale.
P.
h = 26.
2"
A. B. C. D. E. F. G. H. K. L.
M. N. O. P. Q. R. S. T. U. V. W.
a. b. c. d. e. f. g. h. k. l.
m. n. o.w. q. r. s. u. v.

Fig. 386.

ately as shown in Fig. 387, and adding the values of the members that are common to both diagrams. This process is known as the method of superposition. The redundant

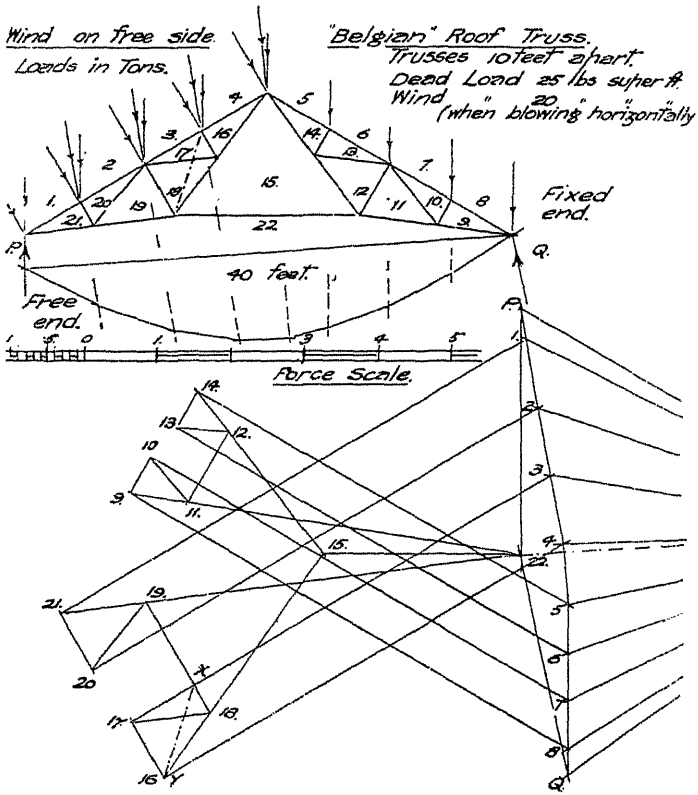


Fig. 388.

members are shown to be 14—15, 13—22, 11—12, 13—23. The following tables give a list of the stresses. No. 1 table gives the results from the combined diagram. No. 2 table from the separated diagrams. The + sign indicates compression, the — sign indicates tension.

TABLE 1. COMBINED DIAGRAM.

TABLE 2. SEPARATED DIAGRAMS.

Bars.		Stress.	Bars.		No. 1 Diagram	No. 2 Diagram	Total.
8–9 7–25	18–17 19–20	+42·5	A–K	D–E	+42·5	—	+42·5
8–7 7–26	18–19 17–20	−28·2	V–W	R–W	—	−28·2	−28·2
9–10 24–11	17–16 21–15	−14·1	H–K	F–E	−14·1	—	−14·1
10–11 9–24	17–21 16–15	+28·2	V–U	R–S	—	+28·2	+28·2
11–12 23–13	15–14 22–13	—	U–T	T–S	redund	ant me	mbers
12–13 11–23	15–22 14–13	+14·1	H–G	F–G	+14·1	—	+14·1
1–18	6–8	+20	N–V	P–R	—	+20	+20
2–16	5–10	+60	N–V P–R	B–H C–F	+20	+40	+60
3–14	4–12	+80	C–F B–H	O–T	+40	+40	+80
2–26	25–26	−30	K–L E–L	—	−30	—	−30
21–26	24–26	−70	K–L E–L	U–W S–W	−30	−40	−70
22–26	23–26	−90	G–L	U–W S–W	−50	−40	−90
P–19	Q–7	+30	—	—	—	+30	+30

EXAMPLE.—Determine the stresses on the bars of a Belgian roof truss of 40 feet span, the trusses being placed at 10 feet centres. Dead load, 25 lbs. per square foot. Wind load, 20 lbs. per square foot normal to the surface. One end fixed, the other end on rollers (see Fig. 389).

Working as before, the dead load comes to ·64 ton on the central points, and 0·32 ton on the end points. The wind load amounts to ·34 ton on the intermediate points and ·17 on the end points. Combine the wind and dead roads on the windward side, by the parallelogram of forces

and determine the resultant. Then proceed (Fig. 389) to draw the force polygon P 1 2 3 4 5 6 7 8 Q. The reaction under the rollers will be vertical. To determine the

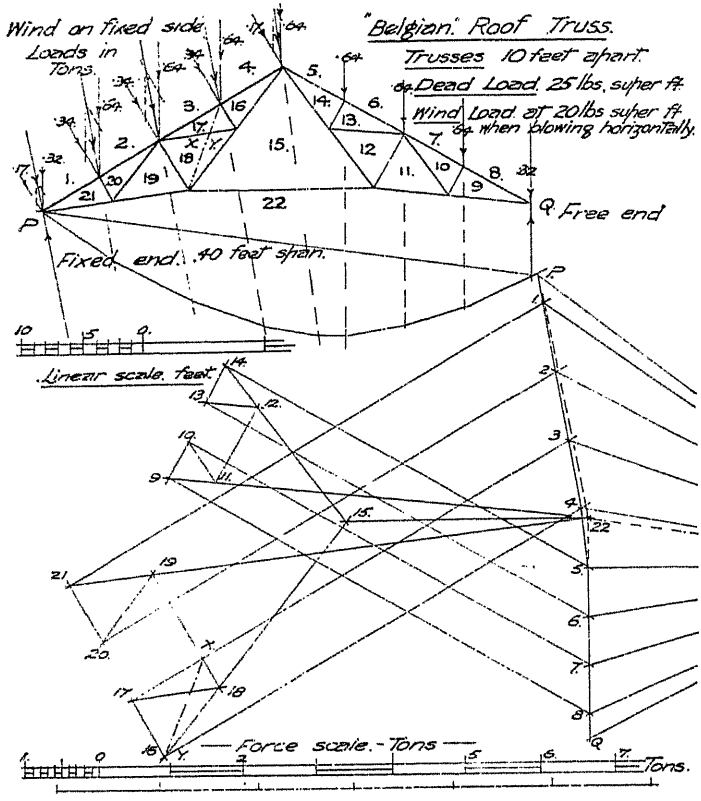


Fig. 389.

magnitude of the reactions and the direction of that at P, draw a polar and funicular or link polygon. The parallel to the closing line on the latter, drawn from the point O on the polar polygon, will determine the limit of the vertical

reaction point 22 ; the line drawn from point 22 to point P will give the magnitudes and direction of the reaction at P, and completes the force polygon. The reciprocal of the frame diagram may now be drawn. There will be a difficulty in determining the point 16—17 and 18, but if the bar XY shown dotted on the frame diagram be substituted for 17—18 and 16—17, the bars about the nucleus 15—Y, Y—X, X—19, 19—22 and 22—15 can be completed. Having obtained the point Y, the point 15 can be found, and replacing the original members, the stresses about the nucleus, 15—16, 16—17, 17—18, 18—15 can be drawn, and the diagram can be completed. Where one end is on rollers, the stresses should be determined when the wind is on the roller side as shown in Fig. 388, as in some cases there will be a considerable difference in the loads on the bars. The maximum stresses obtained under any possible system of loading should of course be taken for purposes of computing the members of the truss.

The recommendations of the B.S.S. for wind stress on roofs ask for a superload of 15 lbs. normal to the slope of the roof on the windward side, and for a suction stress of 10 lbs. on the leeward side. This slightly alters the vector diagram.

EXAMPLE.—Given an arched truss of 60 feet span. Pitch $\frac{1}{3}$. Trusses at 12 feet centres. Dead load 25 lbs. per square foot. Wind normal to slope 15 lbs. per square foot on windward side and 10 lbs. per square foot on the leeward side in an upward direction (see Fig. 390).

Point dead loads	$\frac{2 \times 36 \times 12 \times 25}{10 \times 2240}$	= say 1 ton
Point wind loads	$\frac{36 \times 12 \times 15}{5 \times 2240}$	= say ·6 ton
Do. on leeward side	$\cdot 6 \times \frac{2}{3}$	= ·4 ton.

Then resolve the wind and dead loads. At the nucleus F 11 10 G the load will be the resultant of three forces : 1·0 ton vertical, ·3 ton normal, and ·2 ton normal and in an upward direction to the leeward principal. The point

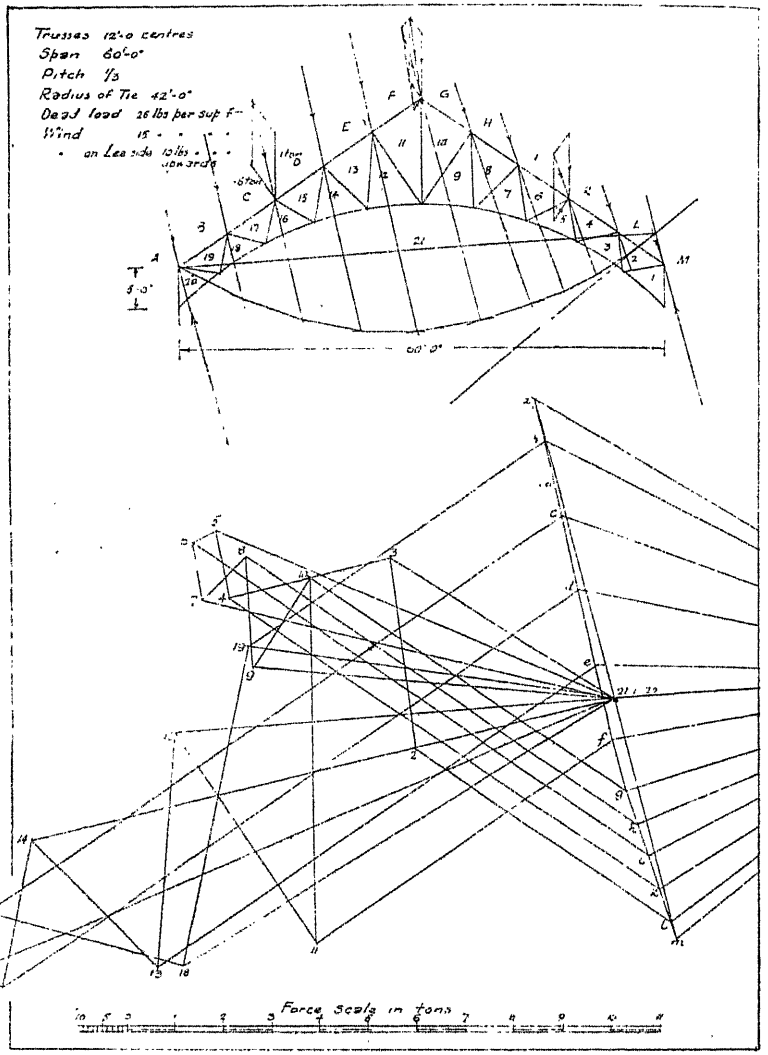

Trusses 12'-0 centres
Span 60'-0"
Pitch 1/3
Radius of Tie 42'-0"
Dead load 26 lbs per sup ft
Wind 15 " " "
" on Lee side 10 lbs " "
upon 3rds
A
T
5'-0"
60' 0"
Force Scale in Tons

Fig. 390.

load on the leeward principal will be the resultant of the 1·0 ton vertical load, and the ·4 ton upward normal.

It will be found that there is no force on the bars A—20, 20—21, nor on the bars 21—1 and 1—M. Proceed as before to set out the vector polygon *a*.....*m* by drawing parallels to the external forces shown on the space diagram. Close the vector polygon by drawing the line *m* to *a*. Proceed with the polar polygon to determine the point 21, thus giving the reactions *m*—21 and 21—*a*. Then complete the stress diagram by drawing parallels to the bars of the roof from the known points on the vector diagram.

CHAPTER XVIII

COLUMNS AND STRUTS

COMPRESSIONAL members, whether of wood, metal, or stone, are termed pillars or columns if placed vertically and when inclined struts.

The failure of columns by crushing occurs only when the length is very small; the sectional area of such members may be determined by the formula

$$A = \frac{P}{p}.$$

Where A = Sectional area.
P = Total load
p = Pressure per unit of area.

Columns whose lengths are great compared with their other dimensions tend to fail by bending, and by combined bending and crushing if of medium length; the bending takes place in the direction of the least radius of gyration r.

$$r^2 = \frac{I}{A}.$$

No very satisfactory theory has yet been evolved or formula devised for determining the strength of columns.

A formula deduced mathematically by Euler, based on the theory of the elastic resistance of materials, is given for struts, the lengths of which are great compared with their least lateral dimensions, *i.e.* :—

$$P = \frac{\pi^2 E I}{l^2}$$

$$\text{and } p = \frac{\pi^2 E r^2}{l^2}.$$

This formula gives fairly accurate results for compres-

sional members, the ratio of length to the least radius of gyration $\left(\frac{l}{r}\right)$ of which is not less than 150. It gives results nearly equal to the full value of a short strut where $\left(\frac{l}{r}\right) = 80$. It gives excessive values where $\left(\frac{l}{r}\right)$ is less than 150.

Euler's formula is a mathematical deduction, based on the assumption of ideal conditions, *i.e.*, the load perfectly axial, uniformity of section, perfect straightness, and ends free to rotate.

These conditions are seldom if ever realized in practice.

Rankine's Formula.—Euler's formula is based on resistance to bending, but in short columns a considerable portion of the resistance is due to the compressional resistance of the material. A modification of Euler's formula was devised by Rankine, which would give approximately accurate results for both long and short columns, and which when the length approached the lateral dimensions would give a value for $p = f_c$, and when the length is great compared with the lateral dimensions approaches the value given by Euler.

p can be considered as the allowable stress on the column under the particular circumstances of length and load, and f_c the safe working stress of the material of the column in compression.

The value of p in Rankine's formula is

$$p = \frac{fc}{1 + \frac{f_c l^2}{\pi^2 \mathrm{E} r^2}}.$$

Where the length is great, the 1 in the denominator becomes negligible and the expression is identical with Euler's. When the length is small, the term $\frac{f_c l^2}{\pi^2 \mathrm{E} r^2}$ becomes negligible and the value of $p = f_c$.

For any given material the expression $\frac{f_c}{\pi^2 \mathrm{E}}$ is a constant quantity and is usually represented by the symbol (a).

The statement then becomes

$$p = \frac{f_c}{1 + a\left(\frac{l}{r}\right)^2}.$$

The expression $\left(\frac{l}{r}\right)$ is known as the buckling factor.

The theoretical values for the factor (a) may be computed for the different materials from the data given, *i.e.*, $\frac{f_c}{\pi^2 E}$, but they are more usually and more satisfactorily determined experimentally (see table opposite).

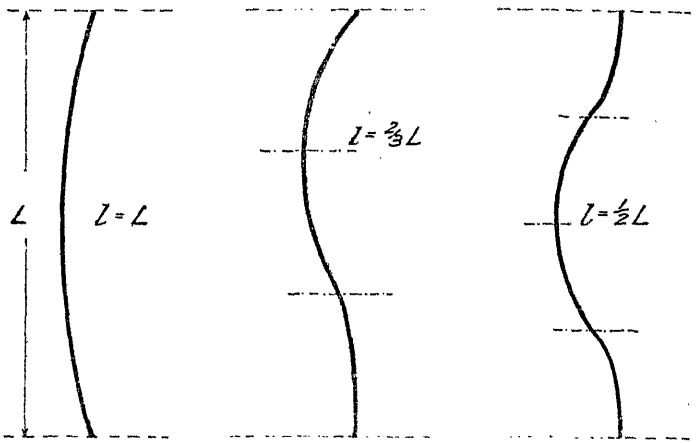


Fig. 391.

The above Fig. 391 shows the three principal conditions affecting the resistances of struts depending upon the end fixings: (1) Ends rounded or free to rotate; (2) one end free to rotate, the other end fixed; (3) both ends fixed. Euler's formula is based on the first of these conditions.

It will be seen from the diagram that case (2) is equal to a column two-thirds of the length of that in case (1), therefore $a_2 = a \times \frac{4}{9}$. In case (3) the length is equal to half the length of case (1) and $a_3 = a \times \frac{1}{4}$.

The following table gives average values of (a) for the common materials of construction.

Material.	Value of f_c in tons./sq. in.	Values of a.		
		Ends free to rotate.	One end Hinged. ,, ,, Fixed.	Both ends Fixed.
Mild Steel ...	7	$\frac{1}{7500}$	$\frac{1}{15000}$	$\frac{1}{30000}$
Wrought Iron	4	$\frac{1}{8500}$	$\frac{1}{17000}$	$\frac{1}{34000}$
Cast Iron ...	6	$\frac{1}{1800}$	$\frac{1}{4000}$	$\frac{1}{7200}$
Timber, Northern Pine	0·7	$\frac{1}{2000}$	$\frac{1}{4500}$	$\frac{1}{8000}$

The factors of safety usually employed for Steel and Wrought Iron are $\frac{1}{4}$th, for Cast Iron $\frac{1}{6}$th, and for Timber $\frac{1}{5}$th.

These formulæ are not suitable for determining directly the dimensions of columns from the known data. It is usual to assume a column and ascertain its resistance. This is not difficult or inconvenient if the values of p for a graduated series of values of $\left(\frac{l}{r}\right)$ are worked out and tabulated, or if a curve of the values be drawn for ratios of $\left(\frac{l}{r}\right)$ between the limits of 0 and 200, this would include all cases likely to occur in ordinary construction.

The following curves show the values for mild steel for two cases of end fixing, worked from the data in the preceding table according to Rankine's formula.

Other Column Formulæ.—In view of the fact that any formula used to obtain a safe working stress on a column is, to a certain extent, arbitrary, it follows that there have been numerous formulæ produced from time to time. The British Standard Specification for the Use of Structural Steel in Building and the London County Council Bye-Laws

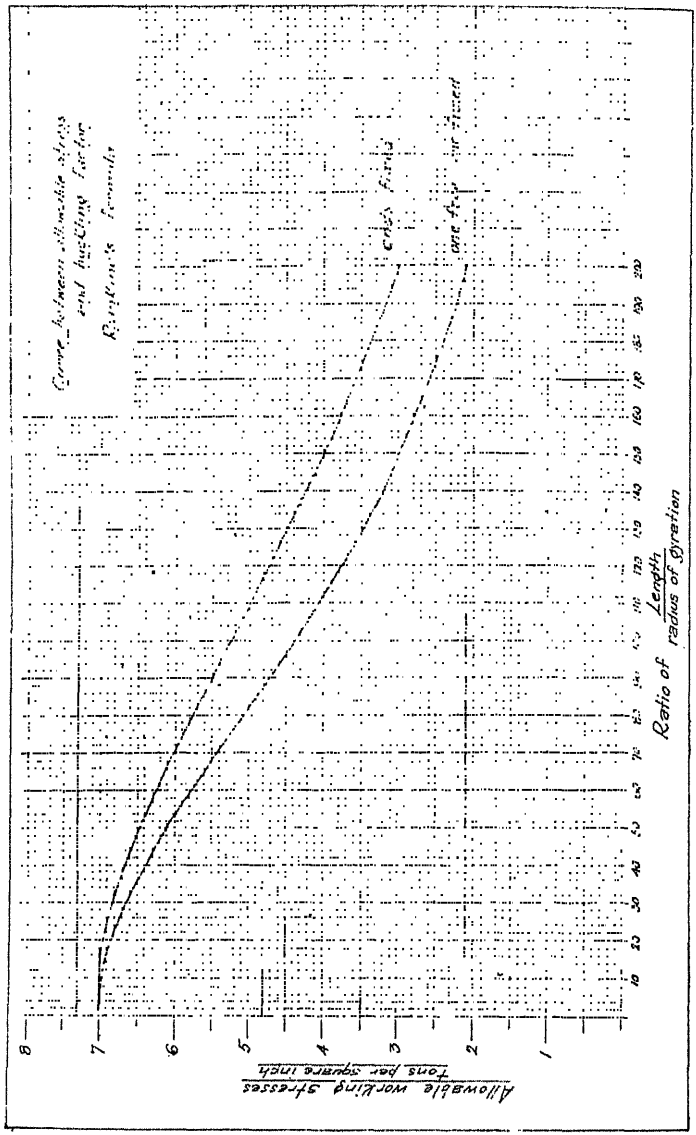

Allowable working stresses
tons per square inch
Ratio of Length / radius of gyration
Curve between allowable stress and buckling factor
Rankine's formula
ends fixed
one end free

Fig. 392.

are similar and the following extracts are from these authorities.

Working Stresses on Columns.—The following paragraphs taken from B.S.S. 449 give the working stresses on columns.

8b. *Reduction of Superimposed Load.* For the purpose of calculating the total load to be carried on foundations, columns, piers and walls in buildings of more than two storeys in height, the superimposed loads for the roof and topmost storey shall be calculated on the loads given in Clause 8a, but for the lower storeys a reduction of the superimposed loads may be allowed in accordance with the following table :—

	Per cent. reduction of superimposed load.
Next storey below topmost storey	10
Next storey below	20
Next storey below	30
Next storey below	40
All succeeding storeys...	50

The above reductions may be made by estimating the proportion of floor area carried by each foundation, column, pier and wall. No such reductions shall be allowed on any floor scheduled for an applied loading of 100 lbs. or more per square foot.

15. (*a*) *Working Stresses in Columns.*—The permissible ratio of effective column length to least radius of gyration shall not exceed the following values :—

(i) For all columns and for struts forming part of the main structure of a building 150

(ii) For subsidiary members in compression ... 240

(*b*) The working stresses per square inch in the shafts of columns and other compression members shall not exceed those specified in the following table (see Appendices A and B) except as provided in Clauses 17 and 18.

Ratio of effective column length to least radius of gyration. $\frac{l}{r}$	Working stresses in tons per sq. in. of gross section. F_1		Ratio of effective column length to least radius of gyration. $\frac{l}{r}$	Working stresses in tons per sq. in. of gross section. F_1	
	Mild steel. B.S.S. No. 15—1936.	High tensile steel. B.S.S. No. 548—1934.		Mild steel. B.S.S. No. 15—1936.	High tensile steel. B.S.S. No. 548—1934.
20	7·17	10·50	130	2·58	2·78
30	6·92	10·11	140	2·28	2·42
40	6·64	9·66	150	2·02	2·12
50	6·30	9·08	160	1·81	1·89
60	5·89	8·34	170	1.62	1·69
70	5·41	7·41	180	1·46	1·52
80	4·88	6·35	190	1·33	1·37
90	4·33	5·32	200	1·21	1·24
100	3·81	4·45	210	1·10	1·13
110	3·34	3·76	220	1·01	1·04
120	2·93	3·21	230	0·93	0·95
			240	0·86	0·87

NOTE.—Intermediate values may be obtained by interpolation.

Effective Column Length.—The effective column length to be assumed in determining the working load per square inch (as set out in L.C.C. Bye-Laws) shall be as follows :—

Type of column.		Effective column length.
Column of one storey.	Adequately restrained at both ends in position and direction.	0·75 of the actual column length.
	Adequately restrained at both ends in position but not in direction.	Actual column length.
	Adequately restrained at one end in position and direction and imperfectly restrained in both position and direction at the other end.	A value intermediate between the actual column length and twice that length, depending upon the efficiency of the imperfect restraint.

Type of column.		Effective column length.
Column continuing through two or more storeys.	Adequately restrained at both ends in position and direction.	0·75 of the distance from floor level to floor level.
	Adequately restrained at both ends in position and imperfectly restrained in direction at one or both ends.	A value intermediate between 0·75 and 1·00 of the distance from floor level to floor (or roof) level, depending upon the efficiency of the directional restraint.
	Adequately restrained at one end in position and direction and imperfectly restrained in both position and direction at the other end.	A value intermediate between the distance from floor level to floor (or roof) level and twice that distance, depending upon the efficiency of the imperfect restraint.

NOTE.—The effective column length values given above are in respect of typical cases only, and embody the general principles which should be employed in assessing the appropriate value for any particular pillar.

The curve (Fig. 393) shows the values of F_1 for values of $\left(\frac{l}{r}\right)$ between the limits of 20 and 200 plotted from the table given in B.S.S. 449.

The L.C.C. curve is identical.

It will be noted that the B.S.S. figures give a flat curve.

A curve of simpler form suitable for all purposes of practical steelwork design is shown by the dotted straight line. The straight line passes through the B.S.S. curve at the points where $\left(\frac{l}{r}\right)$ equals 20, 150 and 200. It changes slope at the value $\left(\frac{l}{r}\right)$ equals 150.

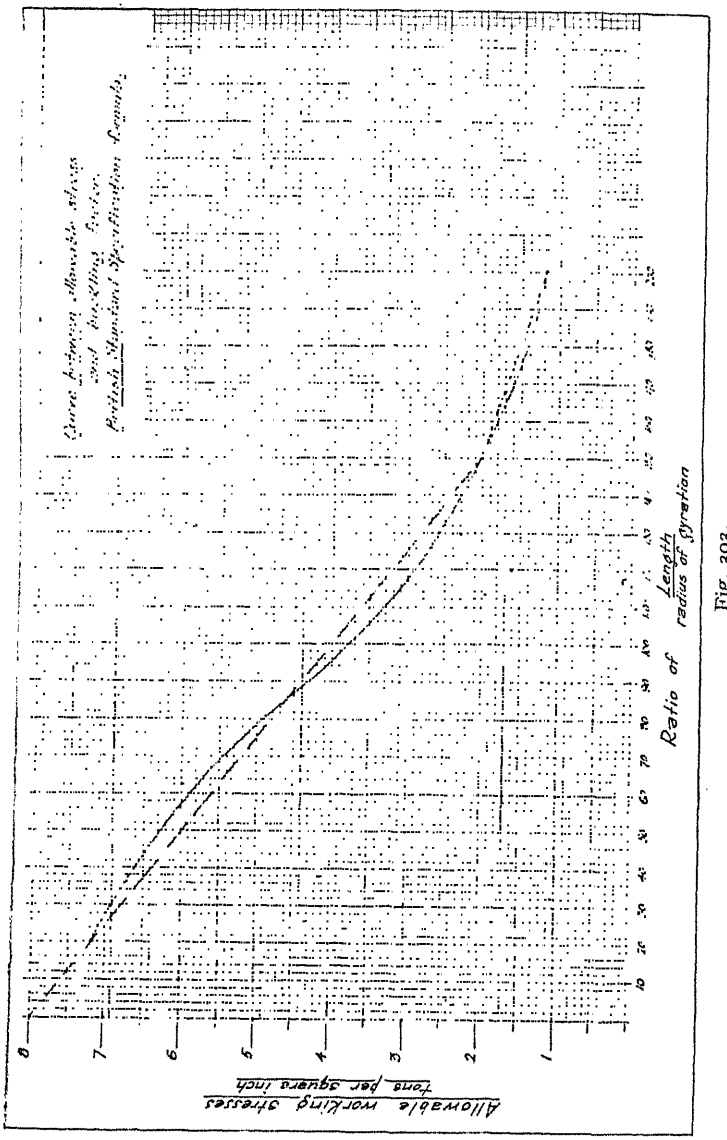

Allowable working stresses
tons per square inch
Ratio of Length / radius of gyration
Curve for steep Allowable stresses
used in Steel Highway Bridges
Portland Standard Specification Formula

Fig. 393.

EXAMPLE.—In Fig. 394 is shown a diagrammatic sketch of a continuous stanchion three storeys high and having four-way connections at the first and second floor and roof levels.

With Rankine Formula.—Assume top and bottom lengths "one hinged and one fixed"; centre length "fixed both ends."

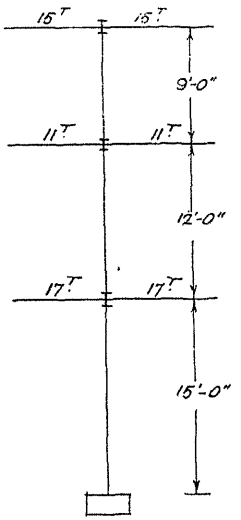


Fig. 394.

With L.C.C. Formula.—Assume centre length "adequately restrained at both ends in position and direction"; top and bottom lengths "adequately restrained at both ends in position and imperfectly restrained in direction at one or both ends."

Use the B.S.S. curve (Fig. 393).

Top Stanchion.—Try 6" × 5" × 25 lbs.

$$\frac{l}{r} = \frac{9 \times 12}{1{\cdot}16} = 93$$

$\therefore$ Allowable stress $F_1 = 4{\cdot}15$ tons/in.^{2} (from curve)

Area (A) $= 7{\cdot}35$ in.^{2}

$\therefore$ Safe load $= 7{\cdot}35 \times 4{\cdot}15$

$= 30{\cdot}5$ tons.

Actual load is 30 tons.

$\therefore$ 6″ × 5″ × 25 lbs. R.S.J. is suitable.

Middle Stanchion.—Try 8″ × 6″ × 35 lbs.

The effective column length is here 0·75 times the actual length.

Effective column length $= 12 \times 0{\cdot}75$

$= 9$ feet.

$$\frac{l}{r} = \frac{9 \times 12}{1{\cdot}38} = 78$$

$\therefore$ $F_1 = 5$ tons/in.^{2}

$A = 10{\cdot}3$ in.^{2}

$\therefore$ Safe load $= 10{\cdot}3 \times 5$

$= 51{\cdot}5$ tons.

Actual load is 52 tons.

$\therefore$ 8″ × 6″ × 35 lbs. R.S.J. is suitable.

Bottom Stanchion.—A larger section than for the middle section will be required. It is sometimes preferable to add plates to the previous section rather than to change to a different larger section. It avoids a joint at the floor level.

Try 8″ × 6″ R.S.J. with 2/9½ × ½″ plates.

$$\frac{l}{r} = \frac{15 \times 12}{2{\cdot}04} = 88.$$

$\therefore$ $F_1 = 4{\cdot}45$ tons/in.^{2}

$A = 19{\cdot}3$ in.^{2}

$\therefore$ Safe load $= 19{\cdot}3 \times 4{\cdot}45 = 86$ tons.

Actual load is 86 tons.

$\therefore$ 8″ × 6″ R.S.J. with 2/9 × ½″ plates is suitable.

If Rankine's formula is used, an exactly similar method is followed, and the size might or might not vary. The only difference in method would be that in the middle stanchion the full length of 12 feet would be taken in calculating $\left(\frac{l}{r}\right)$ and the value of p, corresponding to F_1, read off the curve for " both ends fixed."

EXAMPLE.—What axial load could be carried safely on a 4" × 4" wood post (northern pine), height 8 feet ?

Assume that the end fixing in this case is "one free one fixed." Having no curves for wood columns, Rankine's formula has to be worked out in full. Take $f = 0{\cdot}5$ tons/inch^2, $a = \frac{1}{4500}$

Determine radius of gyration r

$$r^2 = \frac{I}{A} = \frac{bd^3}{12bd} = \frac{d^2}{12}$$

$$\therefore \left(\frac{l}{r}\right)^2 = \frac{l^2}{r^2} = \frac{(8 \times 12)^2}{\frac{d^2}{12}}$$

$$= \frac{(8 \times 12)^2 \times 12}{4^2} = 6900.$$

Rankine's Formula.—

$$p = \frac{f_c}{1 + a\left(\frac{l}{r}\right)^2}$$

$$= \frac{0{\cdot}7}{1 + \frac{6900}{4500}}$$

$$= 0{\cdot}28 \text{ ton/in.}^2$$

Area of post $= 4 \times 4 = 16$ in.^{2}

$\therefore$ Safe load $= 16 \times 0{\cdot}28$

$= 4{\cdot}5$ tons.

Eccentric Loading.—The preceding cases assume that the loads are applied axially. In by far the greater number of cases the loads are otherwise applied. In such cases as the angle stanchion or the stanchions on the front or back faces of buildings, the girders are attached on one, two or three faces of the stanchion respectively, thus inducing a bending stress. It is only in the case of a pair or two pairs of symmetrical loads on opposite faces of the stanchion that the load can be considered as axial. If the resultant of all the loads on a stanchion does not coincide with the axis, then its distance from the axis is termed the arm of eccentricity usually denoted by the letter e, and the bending

moment caused by the product of the load multiplied by the arm of eccentricity has the effect of increasing the compression on the side of the stanchion where the load is applied, and decreasing it on the side remote from the load.

Consider the connection shown in Fig. 395. An angle of 4 inches projection is commonly employed, and the eccentric load P_e is assumed to be acting at a point 2 inches

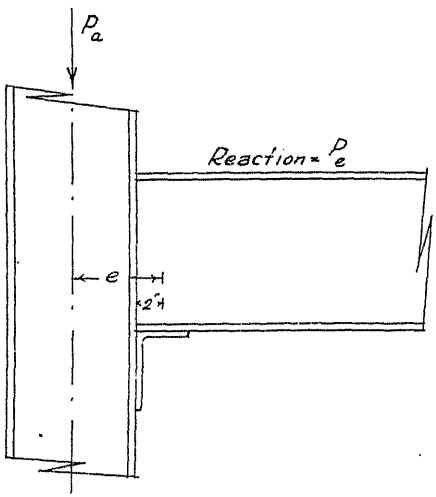


Fig. 395.

from the flange of the stanchion. The value of e is then equal to half the depth of stanchion plus 2 inches. If the girder rests on a bracket fixed to the web the eccentricity is taken as 2 inches.

Let p = total stress due to the axial load plus the stress due to the bending moment produced by the eccentricity of the load. Stress due to bending moment

$$= \frac{My}{I} \left(\text{from } \frac{M}{I} = \frac{f}{y} \right)$$

Bending moment due to eccentric load P_e

$$= P_e \cdot e$$

Maximum total stress due to eccentric load

$$= \frac{P_e}{A} + \frac{P_e \cdot e \cdot y}{I} \qquad \left(\frac{y}{I} = \frac{1}{Z}\right)$$

$$= \frac{P_e}{A} + \frac{P_e \cdot e}{Z}$$

Then if any further axial load be represented by P_a and area of stanchion be A

$$\therefore \text{Maximum stress } p = \frac{P_a}{A} + \frac{P_e}{A} + \frac{P_e \cdot e}{Z}.$$

The above formula is in the form employed for steelwork where values of the modulus of section Z are tabulated. For other materials and for piers, etc., the variation given on p. 292 is used.

The following is an extract from the British Standard Specification No. 449 :—

17. (i) *Eccentric Loading on Columns.*—In the case of columns having loading eccentric to the axis and parallel therewith, the bending moment about each principal axis shall be calculated with proper regard for the eccentricity of the loading, and the maximum compressive stress at the extreme fibre due to the bending actions shall be added to the axial loading per square inch. The sum of these stresses at the extreme fibre shall not exceed F_2 (see Fig. 396 for mild steel).

where : $F_2 = f_c + C\left(1 - \frac{f_c}{F_1}\right)\left(1 - \cdot 002\frac{l}{r}\right)$.

F_1 = the working stress per square inch specified in Clause 15.

f_c = the total axial loading on the column in tons divided by the gross cross sectional area of the column in square inches.

$\frac{l}{r}$ = ratic of effective column length to least radius of gyration.

C = 7·5 in the case of mild steel, 11·25 in the case of high tensile steel.

(ii) In cases where a beam is connected to a continuing column, the bending moment in the column due to the eccen-

tricity of the reaction from the beam may be regarded as divided between the column lengths above and below the level of the beam proportionately to their stiffnesses $\left(\frac{\text{moment of inertia}}{\text{length}} = \frac{I}{l}\right)$, account being taken of all bending or shearing forces at any joint.

(iii) In continuing columns all bending moments due to eccentricities of loading at any one floor level may be disregarded at the levels of the floor beams immediately above and below, provided that the column at these latter

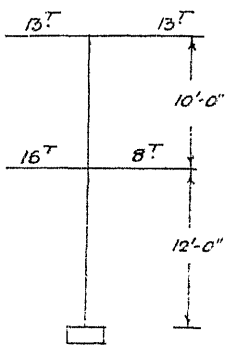


Fig. 397.

levels is effectively restrained in relation to the eccentric load.

EXAMPLE.—Let two loads of 20 and 8 tons be situated on opposite sides of a stanchion at distances of 8 inches and 6 inches respectively from the axis. Then the eccentricity e of the combined loads will be

$$e = \frac{W_1 e_1 - W_2 e_2}{W_1 + W_2}$$
$$= \frac{20 \cdot 8 - 8 \cdot 6}{20 + 8}$$
$$= 4 \text{ inches.}$$

EXAMPLE.—Suggest suitable sizes for the stanchions shown in Fig. 397. Use the B.S.S. formulæ and assume both stanchions to be of the form: "Adequately restrained at both ends in position and imperfectly restrained in

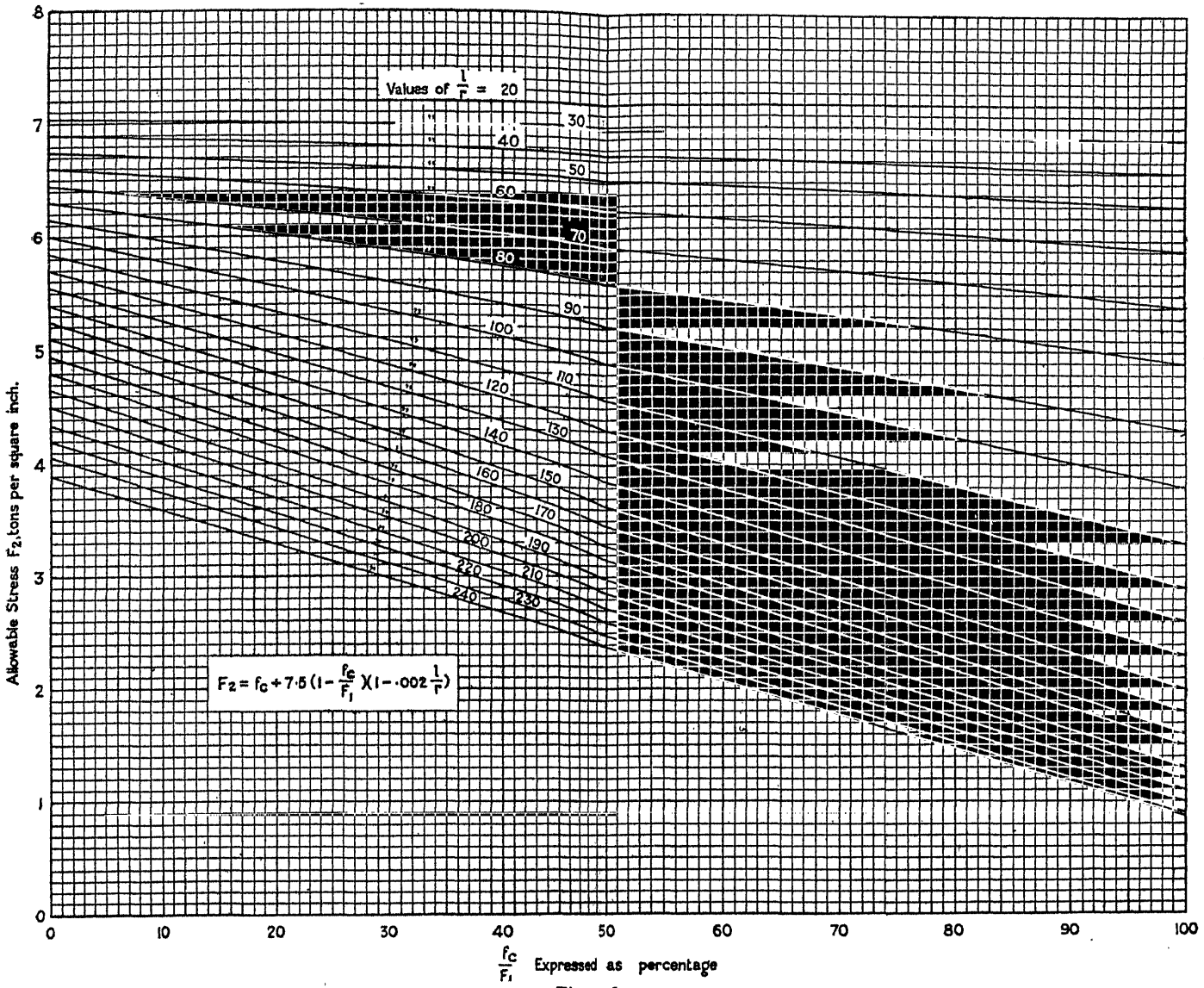


Fig. 396.

[*Between pages* 526 *and* 527.

direction at one or both ends." Main girders are connected to flanges of stanchions.

Assume the reactions shown to include the weights of beams and casings.

Stanchion 1.—Assume 6″ × 5″ × 25 lbs.

Load on stanchion = 26 tons.
Assume pillar casing = 0·7

Total load = 26·7 tons.

$$\frac{l}{r} = \frac{10 \times 12}{1{\cdot}16} = 103.$$

∴ Allowable stress (from curve) = 3·65 tons/in.²

$$\text{Actual stress} = \frac{\text{load}}{\text{area}} = \frac{26{\cdot}7}{7{\cdot}35} = 3{\cdot}68 \text{ tons/in.}^2$$

From the B.S.S. given above, it is seen that there will be a further stress to add due to the eccentricity of the loads at the foot of stanchion 1. It is fairly obvious that this will increase the actual stress above the allowable stress, even when allowance is made for the fact that some small increase in the allowable stress is permissible in accordance the B.S.S.

Try 8″ × 6″ × 35 lbs.

$$\frac{l}{r} = \frac{w \times 12}{1{\cdot}38} = 87$$

∴ Allowable stress = 4·5 tons/in.²

$$\text{Actual stress} = \frac{26{\cdot}7}{10{\cdot}3} = 2{\cdot}59 \text{ tons/in.}^2$$

It would appear that this stanchion is satisfactory, but the above figures will have to be adjusted after stanchion 2 has been calculated.

Stanchion 2.

Axial load from above	= 26·7 tons.
,, ,, girders (2 × 8)	= 16·0 tons.
,, ,, casing (say)	= 1·0 ton.
Total axial load	= 43·7 tons.
Eccentric load	= 8·0 tons.
Total load on foundation	= 51·7 tons.

Try $9'' \times 7'' \times 50$ lbs.

Eccentricity is $(4\frac{1}{2}'' + 2'') = 6\frac{1}{2}$ in.

$$\frac{l}{r} = \frac{12 \times 12}{1{\cdot}65} = 87.$$

Making use of formula in the B.S.S.

$$\frac{f_c}{F_1} = \frac{51{\cdot}7}{14{\cdot}71 \times 4{\cdot}5 \text{ (from curve)}} = 0{\cdot}78.$$

Allowable stress F_2 (from curve) $= 4{\cdot}9$ tons/in.^{2}

Now consider the proportion of bending moment carried by each of the two stanchions due to the eccentric load.

$B = 8$ tons $\times$ 6·5 ins. $= 52$ tons/inches.

$8'' \times 6''$ $\quad \frac{I}{l} = \frac{115}{10 \times 12} = 0{\cdot}96.$

$9'' \times 7''$ $\quad \frac{I}{l} = \frac{208}{12 \times 12} = 1{\cdot}44.$

$1{\cdot}44 + 0{\cdot}96 = 2{\cdot}40$

B taken by $8'' \times 6'' = \frac{52 \times 0{\cdot}96}{2{\cdot}4} = 20{\cdot}7$ tons/inches.

B taken by $9'' \times 7'' = \frac{52 \times 1{\cdot}44}{2{\cdot}4} = 31{\cdot}3$ tons/inches.

Continuing with *stanchion* 2.

$$\text{Actual stress} = \frac{P_a}{A} + \frac{P_e}{A} + \frac{B}{Z}$$

$$= \frac{43{\cdot}7}{14{\cdot}71} + \frac{8}{14{\cdot}71} + \frac{31{\cdot}3}{46{\cdot}3}$$

$= 4{\cdot}19$ tons/in.^{2}

Hence allowable stress $= 4{\cdot}9$ tons/in.^{2}

actual stress $= 4{\cdot}19$ tons/in.^{2}

$9'' \times 7'' \times 50$ lbs. stanchion is suitable.

Go back now to *stanchion* 1. To the actual stress calculated (2·59 tons/inches^2) has to be added the stress due to the proportion of bending moment (20·7 tons/inches) calculated above.

$$\text{Actual stress (revised)} = 2{\cdot}59 + \frac{B}{Z}$$

$$= 2{\cdot}59 + \frac{20{\cdot}7}{28{\cdot}8}$$

$$= 2{\cdot}59 + 0{\cdot}72$$

$= 3{\cdot}31$ tons/in.^{2}

The allowable stress (4·5 tons/inches ^{2}) can be increased, see B.S.S., but in this case it is obviously unnecessary and an $8'' \times 6'' \times 35$ lbs. stanchion is suitable.

It is probable that in practice a $9'' \times 7'' \times 50$ lbs. stanchion would be used from top to bottom to avoid any change in section.

Caps and Bases.—The following is an extract from the regulations given in the British Standard Specification No. 449.

25. *Steel Columns other than Solid Round Steel Columns.*—(*a*) *Foot.*—The foot of every column other than solid round steel columns shall, after riveting up complete with all gussets, stiffeners (if any) base angles and/or cleats, be machined over the whole area of the foot so formed, except as provided by para. (*b*), and shall have affixed thereto either—

(i) A base plate in effective contact with the whole area of the machined foot.

The gusset plates, angles, cleats and stiffeners (if any) in combination with the bearing area of the machined column foot and the base plate shall be sufficient to distribute the load to the foundations without exceeding the stresses given in Clause 10. Rivets transmitting the axial loading need only be capable of transmitting 60 per cent. thereof.

(ii) A slab or bloom base-plate in effective contact with the whole area of the machined foot.

When it can be assumed that the slab distributes the loading uniformly, the minimum thickness, in inches, of a rectangular slab shall be :—

$$t = \sqrt{\frac{3p}{f}\left(y^2 - \frac{y_1^2}{4}\right)}$$

where :

t is the plate thickness in inches,

p is the pressure or loading on base in tons per square inch,

f is the working stress in the steel taken at 9 tons per square inch in the case of mild steel and 13·5 tons per square inch in the case of high tensile steel,

y is the greater projection of plate over column in inches,

y_1 is the lesser projection of plate over column in inches.

When it cannot be assumed that the slab distributes the load uniformly, or where the slab is not rectangular, special calculations shall be made to show that the stresses are within the specified limits.

(*b*) *Ends.*—Except as provided below, each bearing end of each length of all columns other than solid round steel columns shall after riveting up complete with all gussets and end angle cleats, be machined over the whole area of the ends so formed. All joints shall be close-butted and all cap and joint seating plates shall be in effective contact with the whole area of the machined column end.

In cases where sufficient gussets and rivets are provided to transmit the entire loading to the foundations the column ends need not be machined.

(*c*) *Packing.*—The bearing stress in any steel packing or beam interposed between the ends of a superimposed column and the column beneath shall not exceed the stress in the superimposed column, and the width across such interposed steel shall at no point be less than the width of the superimposed column.

(*d*) *Joints.*—All joints in columns shall occur as near as reasonably practicable to floor levels. Joints in columns where bending stresses can produce tension shall be fully spliced to resist such bending. Column joints in which the resultant stress due to all loading and bending moments is wholly compressive shall be sufficiently spliced to retain the members accurately in place, provided that the minimum projection of each splice plate on each side of the column beyond the joint shall be at least equal to the maximum breadth of the column or 8 inches, whichever is the greater.

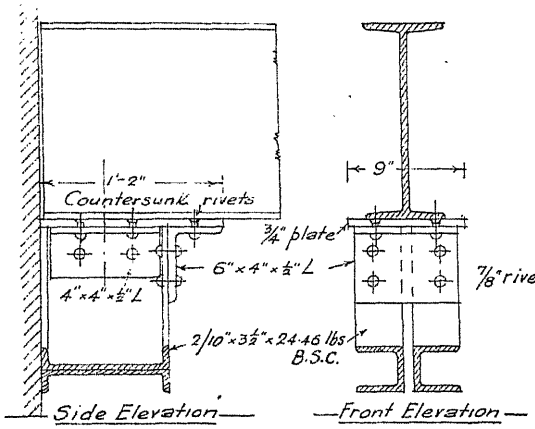


Fig. 398.

Fig. 399.

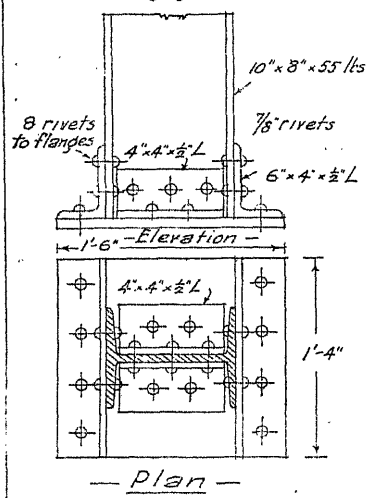


Fig. 400.

Figs. 398—400.

EXAMPLE.—A stanchion consisting of two 10″ × 3½″ × 24·46 lbs. channels supports the end of a girder with a reaction of 24 tons. The stanchion is placed against a wall. The cap consists of a 1′ 2″ × ¾″ plate connected to the shaft by 6″ × 4″ × ½″ angles and four ⅞″ diameter rivets. A ⅞″ diameter rivet has a value in single shear of 3·61 tons. Therefore 4 × 3·61 equals 14·4 tons, which is approximately 60 per cent. of the reaction. Only the front angle is considered, as if there should be any deflection in the girder only the front angle would be affected. (Figs. 398 and 399).

EXAMPLE.—A 10″ × 8″ × 55 lbs. stanchion supports a load of 80 tons. Then from the above the proportion (60 per cent.) of the load taken by the rivets would be 48 tons. Use ⅞″ rivets with values in single and in double shear of 3·61 and 7·22 tons respectively. Let the web be secured by two 4″ × 4″ × ½″ angles, with three rivets in double shear.

Then $48 - (3 \times 7{\cdot}22) = 26{\cdot}4$ tons, and number of rivets required to secure the flanges $= \frac{26{\cdot}4}{3{\cdot}61} = 8$ (Fig. 400).

Joists are connected to other beams or stanchions by cleats, angle brackets, or a combination of both. The principle of construction is that the number of rivets shall be proportional to the pressure transmitted to the connection by the joist to be supported. The rivets must be tested for shearing and bearing, and the number is determined by which of the above two considerations requires the greater area.

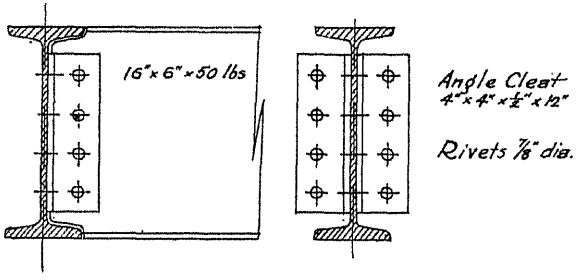


Fig. 401.

The cleats and brackets are standardised and are designed to take the maximum loads for an assumed minimum length for which the girders in question would be likely to be used.

EXAMPLE.—Take a 16″ × 6″ × 50 lbs. R.S.J. (Fig. 401). Take a minimum span of 15 feet giving a maximum reaction of 14 tons. Standard angle cleats are 4″ × 4″ × ½″ × 1′ long, and rivets ⅞ inch diameter. Four ⅞ inch diameter rivets are provided in each cleat for connecting to the web of the main beam, and four rivets in double shear for connecting to the web of the girder. With reference to the safe stresses given in B.S.S. 449, the shop rivets in the smaller girder can be taken at 6 tons/inches^2 or twice this value in double shear, and the field rivets in the connections to the main beam at 5 tons/inches^2. It is probable in practice that the smaller values would be taken throughout.

Value of ⅞″ dia. rivet in single shear at 5 tons/in.^{2}	= 3·01 tons
,, ,, ,, in double shear at (2 × 6) tons/in.^{2}	= 7·22 ,,
,, ,, ,, through web 0·4″ thick at 12 tons/in.^{2}	= 4·2 ,,
Total value connection to main beam, 8 × 3·01	= 24·1 ,,
,, ,, ,, ,, 16″ × 6″ × 50 lbs., 4 × 7·22	= 28·9 ,,
,, bearing in web of 16″ × 6″ × 50 lbs., 4 × 4·2	= 16·2 ,,

Where beams are connected by brackets riveted to the stanchion, the brackets are formed from angle or tee sections for simple connections, or of more complicated combinations for special work, *i.e.*, with wide plate girders formed from combinations of steel joists, etc., or where the reactions are very great. The projection of the brackets is kept small to reduce the eccentricity, unless the bracket is required to form a gusset to stiffen the structure.

Taking the example Fig. 402, the bracket is formed from a 4″ × 4″ × ½″ angle stiffened by a 6″ × 3″ × ½″ tee.

The bracket is usually riveted to the stanchion at the works, and the joist is bolted or riveted to the bracket. To render the connection still further secure an angle cleat is provided ; this is usually riveted to the top flange of the joist and bolted or riveted to the stanchion. The whole of the stress should be taken by the rivets of the supporting bracket. In some cases, for economy, part of the stress is designed to be taken by the rivets in the upper cleat.

For the central stanchions in a frame structure, there will invariably be four such connections on the stanchion, two through the flanges and two through the web.

EXAMPLE.—A 10″ × 8″ × 55 lbs. R.S.J. stanchion has to support two 18″ × 6″ × 55 lbs. joists connected to its flanges with reactions of 18 tons, also two 14″ × 5½″ × 40 lbs. joists connected to its web, with reactions of 9 tons. Let ⅞″ rivets be used, then the B.S.S. value for ⅞″ rivets in single shear at 6 tons/inches² is 3·61 tons, and in double shear at (2 × 6) tons/inches² is 7·22 tons.

Then the number of rivets required for the 18″ × 6″ joists $= \frac{18}{3 \cdot 61} = 5$ rivets, say 6 for symmetry.

The number required for the two 14″ × 5½″ joists $= \frac{18}{7 \cdot 22} = 3$ nearly. See Fig. 405.

Continuous stanchions frequently require to be spliced. The joint should be made just above the girder connections at any storey level. Theoretically, there should be a reduction in the sectional area of the stanchion at every floor level, but in practice it is frequently found more economical to have an excess of metal than to incur the necessity of making a joint at every floor level. Spliced joints are formed by riveting fish plates to the flanges and web of the stanchion.

The ends of the sections to be joined should be planed to obtain good contact in order that the stress may be transmitted directly, as far as possible. As this is not altogether practicable, sufficient rivets must be provided on each side of the joint to transmit the whole of the stress.

EXAMPLE.—A 10″ × 8″ × 55 lbs. R.S.J. 12 feet long has to support a load of 90 tons. Show the fish plates and rivets necessary to form the joint. See Figs. 402 and 403.

$$\frac{l}{r} = \frac{0 \cdot 75 \times 12 \times 12}{1 \cdot 84} = 59$$

∴ From curve allowable stress = 5·9 tons/in.²

Flange area = 8 × 0·783
= 6·26 in.²

Resistance of flange = 5·9 × 6·26
= 37 tons.

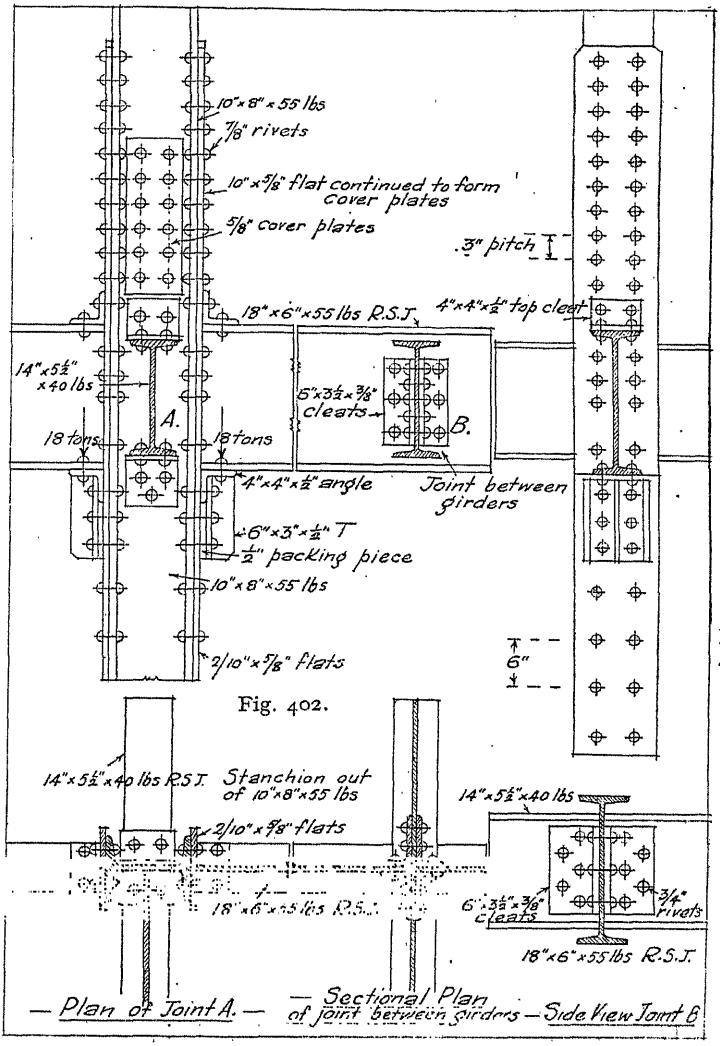


Fig. 402.

Fig. 403.

Fig. 404.

Fig. 405.

Figs. 402—405.

Value of $\frac{7}{8}$" dia. rivet in single shear at 5 tons/in.^{2} = 3·01 tons.

$$\text{No. of rivets required} = \frac{37}{3{\cdot}01} = 12{\cdot}3, \text{ say fourteen rivets.}$$

That is fourteen rivets on each side of the joint on both flanges of the stanchion.

The web area equals, say, $0{\cdot}4\ \{10 - (2 \times 0{\cdot}783)\} = 3{\cdot}37$ in.

Resistance of web = 3·37 × 5·9
= 19·9 tons.

The strength in bearing through the web will govern the number of rivets required.

At 10 tons/inches^2, the resistance in bearing through a web 0·4 inch thick is

$$\tfrac{7}{8} \times 0{\cdot}4 \times 10 = 3{\cdot}5 \text{ tons.}$$

$$\text{No. of rivets required} = \frac{19{\cdot}9}{3{\cdot}5} = \text{six rivets.}$$

That is six rivets on each side of joint in web.

Let the stanchion below the floor level consist of a 10" × 8" × 55 lbs. R.S.J. and two 10" × $\frac{5}{8}$" flats. Then the 10" × $\frac{5}{8}$" flats would be continued to form the covers for the flanges.

Circular Columns.—There is a relatively small demand for columns circular in section, owing to their greatly increased cost, compared to stanchions of equal strength, and also to the greater difficulty in making connections.

The following is an extract from B.S.S. No. 449.

24. *Solid Round Steel Columns.*—Solid round steel columns shall have machined shouldered ends and shall be provided with caps and bases, the bearing surfaces of which shall be machined after being shrunk or screwed on. When it can be assumed that the load on the cap or under the base is uniformly distributed, the minimum thickness, in inches, of a square cap or base shall be

$$\sqrt{\frac{9W}{16f} \cdot \frac{D}{D - d}} \text{ where :—}$$

W is the total axial load in tons,
D is the length of the side of cap or base in inches,
d is the diameter of the reduced end of the column in inches.

f is the working stress in the steel taken at 9 tons per square inch in the case of mild steel and 13·5 tons per square inch in the case of high tensile steel.

When it cannot be assumed that the load on the cap or under the base is uniformly distributed, or where the cap or base is not square, special calculations based on the same working stress shall be made.

The cap or base plate shall not be less than 1·5 $(d + 3)$ inches in length or diameter.

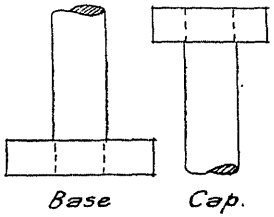


Fig. 406.

EXAMPLE.—Determine the total strength of a steel column 6 inches diameter, 12 feet high. Assume effective length equal to actual length.

$$\frac{l}{r} = \frac{12 \times 12}{1{\cdot}5} = 96.$$

Allowable stress (from curve) = 4 ton/in.²

Allowable load = 4 × area = 4 × 28·27

= 113 tons axial.

It is especially important that the load on a circular column should be placed axially.

EXAMPLE.—Take the column as in previous example and assume that it carries an eccentric load of 30 tons as in Fig. 407. Eccentricity = 3 inches.

$$\frac{l}{r} = \frac{12 \times 12}{1{\cdot}5} = 96.$$

Allowable stress for axial loads = 4 tons/in.²

Modulus of section $Z = \frac{I}{y} = \frac{\pi D^4}{64\,y}$

$$= \frac{\pi \times 6^4}{64 \times 3}$$

$$= 21{\cdot}2 \text{ in.}^3$$

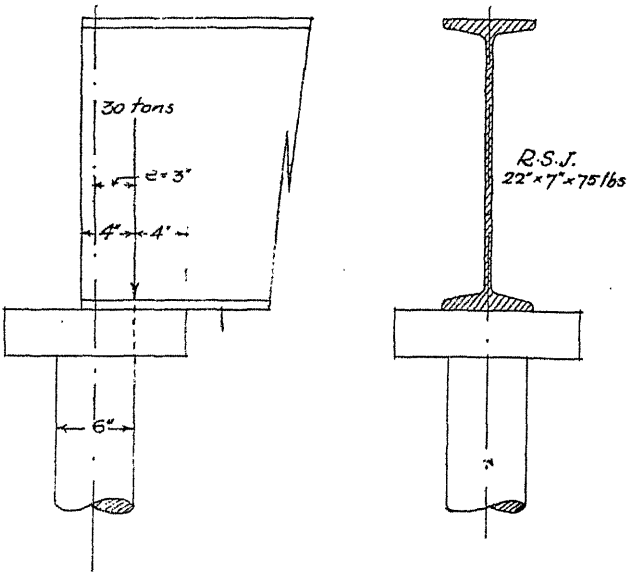


Fig. 407.

Actual stress on column $= \frac{P_e}{A} + \frac{P_e \times e}{Z}$

$$= \frac{30}{28{\cdot}27} + \frac{30 \times 3}{21{\cdot}2}$$

$$= 5{\cdot}3 \text{ tons/in.}^2$$

Making use of the B.S.S. formula.

$$\frac{f_c}{F_1} = \frac{30}{28{\cdot}27 \times 4} = 0{\cdot}265.$$

Then from appropriate curve:—Increased allowable stress = 5·5 tons/inches². Hence it is seen that a 6-inch

column will do. By comparing the above two examples, the big reduction in load carried, due solely to the eccentricity, must be noted.

Wind Stress on Columns.—British Standard Specification No. 449 says :—

Wind.—The design shall allow for a wind pressure in any horizontal direction of not less than 15 lb. per square foot of the upper two-thirds of the vertical projection of the surface of such buildings, with an additional pressure of 10 lb. per square foot upon all projections above the general roof level. On the sea coast and in similarly exposed situations a further provision shall be made.

If the vertical projection of a building is less than twice its width, wind pressure may be neglected, provided that the building is adequately stiffened by floors and walls.

NOTE.—The wind loads stipulated in this clause are such as would be regarded as adequate in Great Britain, but may possibly require modification in other countries.

Stresses Due to Wind Forces.—18.—The working loads and stresses per square inch specified hereinbefore for beams, columns and all their connections as computed for all loads and forces other than wind pressure may be increased by 33⅓ per cent. in cases where such increase is solely due to stresses induced by wind pressure, provided that such increase shall not apply to the stresses given in Clauses 11 and 12.

EXAMPLE.—Take the case shown in Fig. 408 and determine the additional loads and bending moments put on the columns due to the wind pressure. Frames 20 feet centres.

Consider the column B as the centroid of a group of columns. The wind acting on the left-hand side will cause a compression on column C and a tension on column A. In this case column B will be unaffected.

Consider the wind pressure above floor 5. The total pressure P acts at centre of area under consideration.

Let h be the height of P in each case above the floor level.

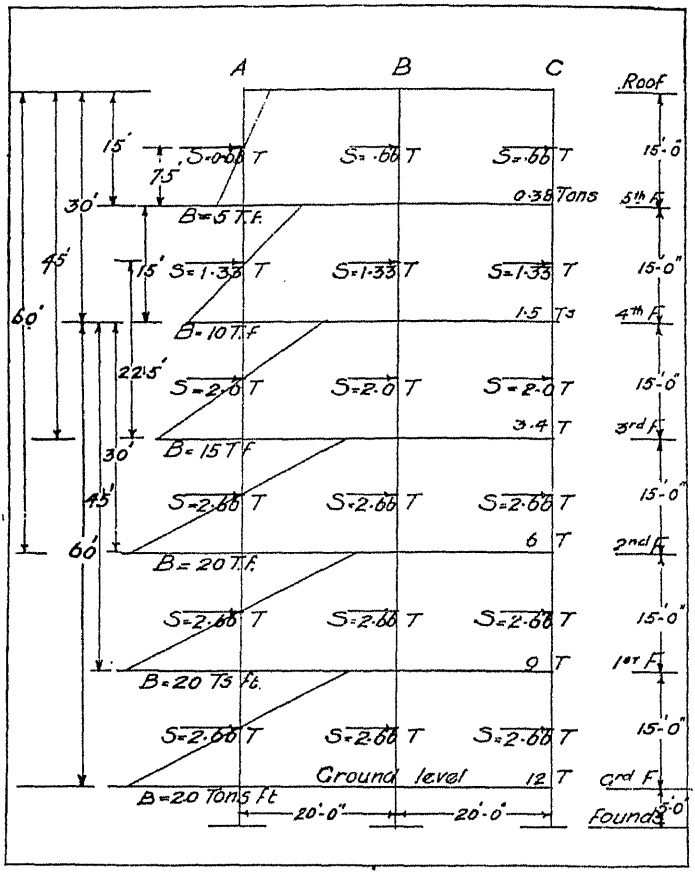


Fig. 408.

Then Ph = moment due to wind load. From $\frac{M}{I} = \frac{f}{y}$ is derived

$$f = \frac{My}{I}$$

The load due to wind in stanchion C will be a point load at a distance y from the centroid, (stanchion B).

$\therefore$ In this case

$$I = Ay^2$$

$$f = \frac{My}{Ay^2} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (1)$$

Let F = total load at distance $y = fA$. $\quad M = Ph$.

$\therefore$ Substituting in (1) above.

$$F = \frac{Phy}{y^2}$$

$$F = \frac{Ph}{y}$$

In this example two columns, A and C, will divide the load.

$$\therefore \quad F = \frac{Ph}{2y}$$

Compression in column on lee side.

D above 5th floor $= 20 \times 15 \times 15 = 4500$ lbs.
say $= 2$ tons.

F at 5th floor level $= \frac{2 \times 7{\cdot}5}{2 \times 20} = 0{\cdot}38$ ton.

P above 4th floor level $= \frac{15 \times 20 \times 30}{2240} = 4$ tons.

F at 4th floor level $= \frac{4 \times 15}{2 \times 20} = 1{\cdot}5$ tons.

P above 3rd floor level $= \frac{15 \times 20 \times 45}{2240} = 6$ tons

F at 3rd floor level $= \frac{6 \times 22{\cdot}5}{2 \times 20} = 3{\cdot}4$ tons.

P above 2nd floor level $= \frac{15 \times 20 \times 60}{2240} = 8$ tons

F at 2nd floor level $= \frac{8 \times 30}{2 \times 20} = 6$ tons.

P above first floor level = 8 tons. This is the same as above second floor level because the wind pressure is considered to act on the upper two-thirds only.

F at 1st floor level $= \frac{3 \times 45}{2 \times 20} = 9{\cdot}0$ tons.

P above ground level $= 8$ tons.

F at ground level $= \frac{8 \times 60}{2 \times 20} = 12$ tons.

Thus with the wind blowing from the left as in Fig. 408, the values of F calculated above are the additional loads in compression placed on stanchion C due to the wind. The loads appear in tension on stanchion A.

Bending moment in columns.

The bending moment due to the wind is calculated on each column by multiplying the horizontal shear or thrust by half the height of the column. As there are three columns the horizontal shear at each floor level will be the values of P as calculated above divided by three. Let S be the horizontal shear.

S at 5th floor level $S_5 = \frac{2}{3} = 0{\cdot}66$ ton.

$S_4 = \frac{4}{3} = 1{\cdot}33$ tons.

$S_3 = \frac{6}{3} = 2$ tons.

$S_2 = \frac{8}{3} = 2{\cdot}66$ tons.

$S_1 = 2{\cdot}66$ tons.

$S_G = 2{\cdot}66$ tons.

Bending moment B at 5th floor level.

$B_5 = 0{\cdot}66 \times \frac{15}{2} = 5$ tons ft.
$B_4 = 1{\cdot}33 \times 7{\cdot}5 = 10$ tons ft.
$B_3 = 2 \times 7{\cdot}5 = 15$ tons ft.
$B_2 = 2{\cdot}66 \times 7{\cdot}5 = 20$ tons ft.
$B_1 = 2{\cdot}66 \times 7{\cdot}5 = 20$ tons ft.
$B_G = 2{\cdot}66 \times 7{\cdot}5 = 20$ tons ft.

Bending moment on beam.

Calculate the bending moment that is impressed on the beams due to the wind pressure. Consider as an example the beam at fourth floor level.

Bending moment at end adjoining column A is

$$B = (1{\cdot}33 \times 7{\cdot}5) + (2 \times 7{\cdot}5)$$
$$= 25 \text{ tons ft.}$$

Bending at end adjoining centre column is

$$B = (1{\cdot}33 \times 7{\cdot}5) + (2 \times 7{\cdot}5) + (1{\cdot}5 \times 20) - (3{\cdot}4 \times 20)$$
$$= 13 \text{ tons ft.}$$

The point of contraflexure occurs at a point 13·2 feet from column A.

These bending moments produced in the beam due to wind must be considered in the design of the beam together with the bending moments usually considered in calculating the size of a girder.

Summary of values obtained :—

Column above floor.	Vertical force. Cols. A and C. Tons (F).	Horizontal shear or thrust. Tons (S).	Maximum bending moment. Tons feet (B).
5	0·38	0·66	5
4	1·5	1·33	10
3	3·4	2·00	15
2	6·0	2·66	20
1	9·0	2·66	20
Ground	12·0	2·66	20

CHAPTER XIX

GIRDERS

It is usual to consider a beam as being a horizontal bar of material upon which act a number of vertical forces. It follows from the fact that any given beam is in equilibrium, that :—

1. The algebraic sum of all the forces acting on the beam must be zero.

2. The algebraic sum of the moments of all the forces about any point must be zero.

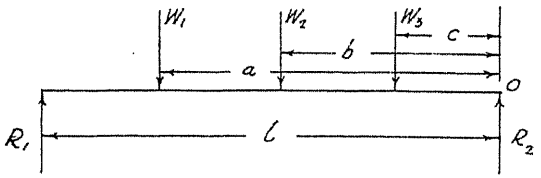


Fig. 409.

In any given case it is necessary to determine the value of the reactions first. Consider the beam represented in outline in Fig. 409. Employing the fact that the algebraic sum of all the moments about any point must be zero, take moments about a point lying in the line of action of one of the reactions, say O.

Then $$R_1 l - W_1 a - W_2 b - W_3 c = 0$$

This is an equation having only the one unknown, namely R_1, and therefore it can be evaluated. By taking moments about a point in the line of action of R_1, an equation for R_2 is obtained, thus :—

$$R_2 l - W_1(l - a) - W_2(l - b) - W_3(l - c) = 0.$$

Alternatively, in view of the fact that

$$R_1 + R_2 = W_1 + W_2 + W_3$$

it is possible to obtain the remaining reaction by subtracting the known reaction from the total load.

Shear Force and Bending Moments.—Consider the beam illustrated in Fig. 410. Consider also a cross section of the

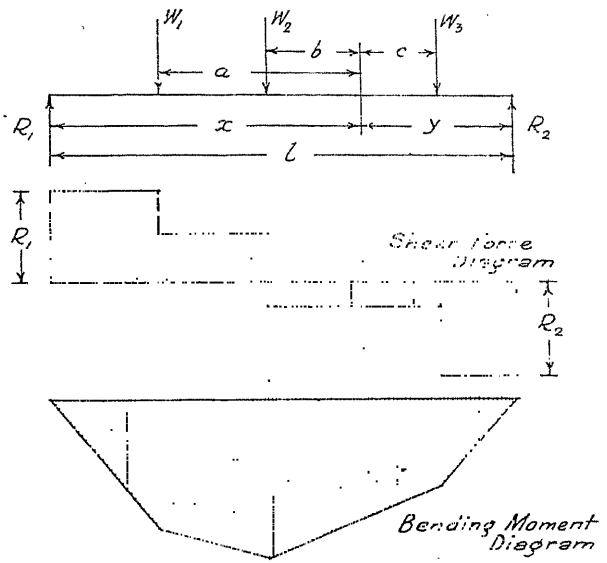


Fig. 410.

beam at a point X, distance x from the left-hand support. The beam as it stands is in equilibrium. Imagine that the beam is divided at section X. The two parts of the beam thus formed would not be in equilibrium. Considering for the time the left-hand section. The rule given above that the algebraic sum of all the forces must be zero, does not hold. Namely, $R_1 - W_1 - W_2$ does not equal zero.

It follows, therefore, that if this left-hand section is to be considered as in equilibrium, there must be introduced a further force F, and

$$F = R_1 - W_1 - W_2.$$

When the beam is considered as a whole this force is supplied at the section X by the shearing resistance of the fibres of the beam material. This force is known as the shear force at the section X.

The above argument could be applied equally to the right-hand section of the beam, the value of the shear force for any given section being the same either way.

Hence the Shear Force at any section in a beam is the algebraic sum of all the forces to the left (or right) of the section.

Referring again to Fig. 410 the bending moment in the beam is considered in a similar manner. Dealing with the left-hand section and imagining that the beam is divided at section X, there is an unbalanced moment when the moments of the three known forces are taken about X.

The second rule given above states that the algebraic sum of the moments of all the forces about any point must be zero. When the beam is considered as a whole, and therefore in equilibrium, this unbalanced moment at X which is known as the bending moment, is balanced by a moment exerted by the right-hand section on the left-hand section. This balancing moment is known as the resistance moment.

Let B = Bending moment
Then $B = R_1x - W_1a - W_2b$.

The value of the bending moment will be the same if considered from the other end of the beam. $- R_1y + W_3c = B$.

Hence the Bending Moment at any section in a beam is the algebraic sum of the moments about that section of all the forces to the left (or right) of the section.

Shear Force and Bending Moment Diagrams.—As the section X in Fig. 410 may be anywhere in the beam it will be seen that the values of the shear force and bending moment vary along the length of the beam. It is customary to determine values for the shear force and bending moment at a number of points along the beam and to plot the results to scale at right angles to a horizontal base line. These diagrams are illustrated in Fig. 410.

Standard Examples.—The following comparative examples illustrate the methods of determining the bending moments and shear forces for the commonest cases in general practice.

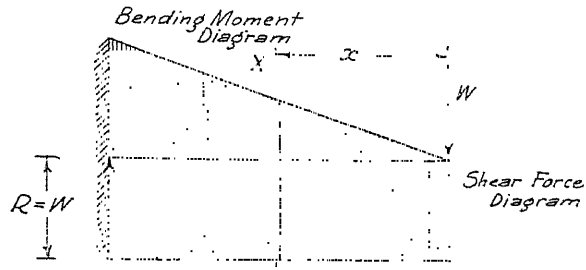


Fig. 411.

Case 1.—Cantilever loaded at free end (Fig. 411).

Shear.—The sum of the vertical forces to the right of any section in the beam equal W and the diagram is a rectangle.

Bending Moment.—

Let B_x = bending moment at any section X.

$B_x = Wx$.

The bending moment varies at any point in the beam as x, the distance along the beam. The diagram, therefore, becomes a triangle, the ordinates of which vary from zero at the free end to a maximum at the support.

Let $B_{max.}$ = maximum bending moment.

$\therefore B_{max.} = Wl$ at the support.

Case 2.—Cantilever under a distributed load.

Let w = load per foot run.

Shear.—The shear force at any section x is as follows.

$$F_x = wx.$$

Taking varying values for x along the beam a triangular diagram is obtained.

$F_{max.} = wl = W$ at the support where $x = l$.

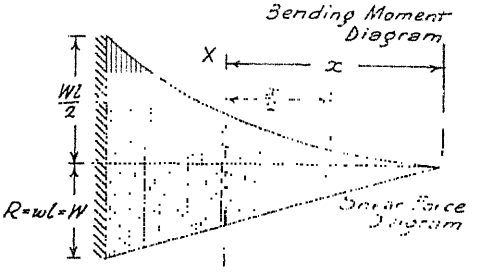


Fig. 412.

Bending Moment.—The bending moment at any section X is as follows :—

$$B_x = wx \times \frac{x}{2} = \frac{wx^2}{2}$$

because the load to the right of the section is wx and the average leverage (taken as the arm of the moment) is $\frac{x}{2}$

$$B_{max.} = \frac{wl^2}{2} = \frac{Wl}{2} \text{ where } x = l$$

The bending moment at any point varies as x^2 and the outline of the diagram is parabolic.

Case 3.—Beam supported at ends with a concentrated load, Fig. 413. Find reactions.

$$R_1 l = Wb$$

$$\therefore R_1 = \frac{Wb}{l}$$

Similarly $$R_2 = \frac{Wa}{l}$$

Shear.—In section a of the beam.

$F = R_1$ (sum of forces on left-hand side of any section X).

In section b of the beam

$$F = R_1 - W$$

Bending Moment.—The bending moment at any section X in section *a* of beam is :—

$$B_x = R_1 x$$

and varies as x.

At any section X in section *b* of beam

$$B_x = R_1 x - W(x - a).$$

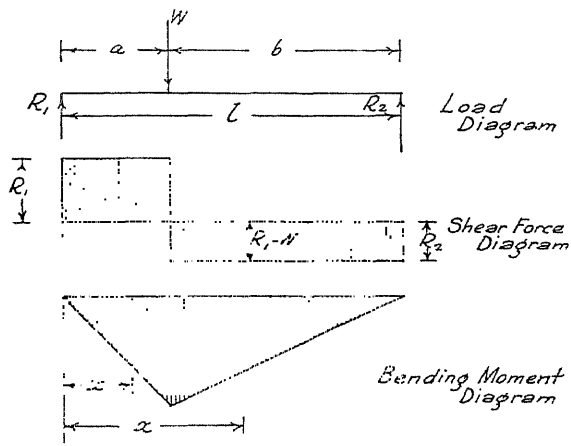


Fig. 413.

From the diagram the maximum bending moment occurs underneath the load when $x = a$.

$$\therefore B_{max.} = R_1 a.$$

Central Load.—If the load is central.

$$R_1 = R_2 = \frac{W}{2}$$

$$F_{max.} = R_1 = R_2 = \frac{W}{2}$$

$$B_{max.} = R_1 a = \frac{W}{2} \cdot \frac{l}{2}$$

$$B_{max.} = \frac{Wl}{4}.$$

Case 4.—Beam supported, load distributed.

Let w = load per foot run.

Shear.—Shear at any section X is

$$F_x = R_1 - wx.$$

This equation for shear involves the first power of the variable x only and therefore gives a straight line diagram.

$$F_{max.} = R_1 = R_2.$$

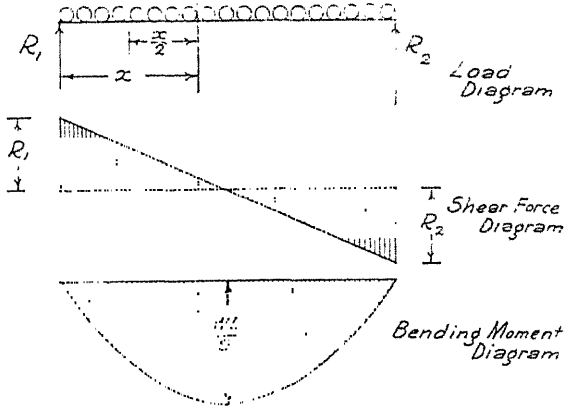


Fig. 414.

Bending Moment.—Bending moment at any section X

$$B_x = R_1 x - \frac{wx^2}{2}.$$

The diagram obtained from the above equation gives a parabolic bending moment diagram as shown.

$$B_{max.} = R_1 \frac{l}{2} - \frac{wl^2}{8} \text{ when } x = \frac{l}{2}$$

$$= \frac{W}{2} \cdot \frac{l}{2} - \frac{Wl}{8}$$

$$B_{max.} = \frac{Wl}{8}.$$

Moment of Resistance.—A beam subjected to a load tends to bend. The resistance to bending sets up varying longitudinal stresses in the beam. The effect of a load on a beam simply supported is to compress the fibres exposed

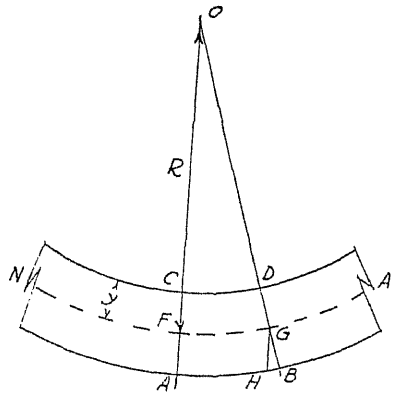


Fig. 415.

to the load and to extend those on the side of the neutral axis remote from the load, Fig. 415. Thus on one side of the neutral axis the fibres are under a crushing stress and on the other side a tensile stress.

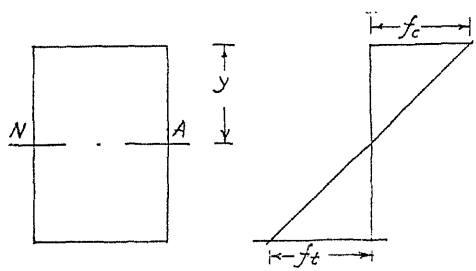


Fig. 416.

In Fig. 416 is shown the stress diagram for a rectangular section and it is seen that the intensity of the stresses is proportional to the distance from the neutral axis at any section. The sum of the product of these stresses multiplied

by their leverages from the neutral axis is a moment which is termed the moment of resistance.

In Fig. 415 let AC and BD be two sections of a beam very close together. Produce the lines until they meet in O.

$$FO = \text{Radius of curvature} = R.$$

From G draw GH parallel to FO.

Then AH = FG, the original length of the fibres in the unbent beam.

Assume HB = f = the stress in the fibres distant y from the neutral axis. The elongation in the fibres is directly proportional to the stress producing it, providing it is within the limit of elasticity.

If E = modulus of elasticity

$$E = \frac{\text{Stress}}{\text{Strain}} = \frac{f}{\frac{HB}{AH}} = \frac{f \times AH}{HB}.$$

As $HB = f$

$\therefore E = AH = FG.$

Then the length $FG = E.$

From similar triangles

$$\frac{HB}{FG} = \frac{HG}{FO}$$

$$\frac{f}{E} = \frac{y}{R}$$

$$\frac{f}{y} = \frac{E}{R} \quad \ldots\ldots\ldots \quad (1)$$

or longitudinal stress is

$$= \frac{Ey}{R}.$$

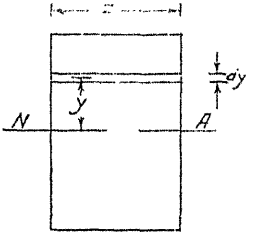


Fig. 417.

Let Fig. 417 be a section of a beam and $z\,dy$ an element of area of the section.

Element of area $= z \,.\, dy$
Longitudinal force on area $= f \,.\, z \,.\, dy$
Moment of force $= f \,.\, z \,.\, y \,.\, dy$

Summing up these moments over the whole section and substituting $f = \frac{Ey}{R}$ from (1)

Then
$$M = \frac{Ey}{R} \Sigma z \,.\, y \,.\, dy$$
$$= \frac{E}{R} \Sigma z \,.\, y^2 \,.\, dy \quad \ldots\ldots\ldots (2)$$

But $\Sigma z\, y^2\, dy$ = the product of the sum of all the elements of area multiplied by their distance squared from the NA
= the moment of inertia of the section about the NA

That is $\Sigma z\, y^2\, dy = I$

From (2)
$$M = \frac{E}{R} \,.\, I$$
$$\therefore \frac{M}{I} = \frac{E}{R} \quad \ldots\ldots\ldots (3)$$

Combining (1) and (3)
$$\frac{M}{I} = \frac{f}{y} = \frac{E}{R} \quad \ldots\ldots\ldots (4)$$

Where
M = moment of resistance of section
I = moment of inertia of section
f = stress at distance y from NA
E = modulus of elasticity of material
R = radius of curvature.

The resistance moment must equal the bending moment at any section.

Thus
$$M = B.$$

In calculations involving the use of equation (4), the value of the bending moment may be substituted for M, the resistance moment.

From equation (4)
$$\frac{M}{f} = \frac{I}{y}$$

The quantity $\frac{I}{y}$ is termed the modulus of the section and is usually denoted by the letter Z.

Thus $$Z = \frac{M}{f}.$$

In the case of unsymmetrical sections the modulus has two values, i.e.—

$$\frac{I}{y_t} = Z_t \text{ and } \frac{I}{y_c} = Z_c.$$

To determine the position of the neutral axis where the value of E is the same in tension and compression.

In the figure 418 let $z.dy$ be a strip distant y from the NA. Then the stress at $y = f_1$.

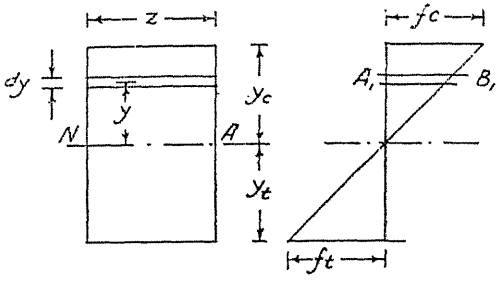


Fig. 418.

But from the figure

$$\frac{A_1B_1}{y} = \frac{f_c}{y_c}$$

and $A_1B_1 = \frac{f_c y}{y_c}$ = the stress in the section $z.dy$.

The total stress in the section above the NA

$$= \Sigma \frac{f_c.y.z.dy}{y_c} = \frac{f_c}{y_c} \Sigma y.z.dy$$

$$= \frac{f_c}{y_c} \times \text{the first moment of the area above the NA about the NA.}$$

Similarly, for the section below the NA

$$\frac{f_t}{y_t} \Sigma y.z.dy = \frac{f_t}{y_t} \times \text{the first moment of the area below the NA about the NA}$$

And as the total tension equals the total compression and is opposite in sense, therefore the total first moment of the whole area about the NA = zero. But the first moment of an area about a line equals zero only when the line passes through the centroid, see p. 475 ; therefore the NA passes through the centroid of the area.

Shearing.—The shearing stress on a beam has already been defined as the algebraic sum of all the vertical forces on either side of a given point on a beam. These vertical forces, acting as couples at any part give rise to horizontal couples equal in magnitude. Consider a small section of material ABCD of unit depth. Let AD and BC represent the vertical couple, and AB, CD the horizontal couple. Then the forces on AD and AB due to the stresses p and p_1 are p.AD and p_1.AB respectively. Equate the moments of the two couples.

$$p \times \text{AD} \times \text{AB} = p_1 \times \text{AB} \times \text{AD}$$

$$\therefore \qquad p = p_1.$$

Thus the vertical shear stress is equal to the horizontal shear stress.

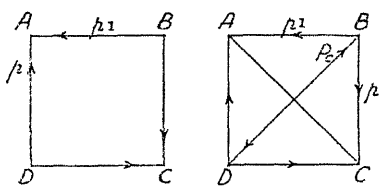


Fig. 419.

The effect of the two couples is to cause tensional and compressional stresses along the diagonal lines, the intensity of which will equal the horizontal and vertical shear stress. Thus in Fig. 419 let P_c represent the force acting along BD. Then

$$P_c^2 = (p.\text{AD})^2 + (p_1.\text{AB})^2$$

$$= p^2.\text{AD}^2 + p_1^2.\text{AB}^2.$$

Now $\quad P_c = p_c \text{BD}$

and $\quad p = p_1.$

Substituting

$$p_c^2.BD^2 = p^2.AD^2 + p^2.AB^2$$
$$= p^2(AD^2 + AB^2)$$
$$= p^2.BD^2$$
$$\therefore \quad p_c = p.$$

Similarly

$$p_t = p.$$

The effect of this in rolled steel beams and plate girders is to cause buckling in the web.

The horizontal shearing stress varies over any transverse section, being a maximum at the NA and a minimum at the fibres most remote from the NA. The method of determining the distribution and the intensity of the shear at any part of the section is shown as follows :—

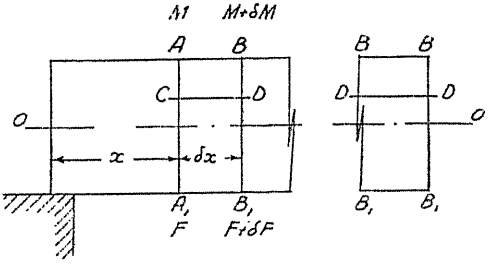


Fig. 420.

Fig. 420 represents a portion of the length of a beam : AA_1 is a section distant x, and BB_1 a section $x + \delta x$ from the end of the beam.

Let the bending moment at A = M and at B = M + δM.

Let the shear force at A = F and at B = F + δF. If we consider the section δx, F acts downwards and F + δF upwards.

If the loading on the section be w per unit length.

Then

$$F + \delta F = F + w\delta x$$
$$\frac{\delta F}{\delta x} = w$$
$$\text{or } \frac{dF}{dx} = w$$

Take moments about a point on the section BB_1

$$(M + \delta m) - F\delta x - \frac{w\delta x^2}{2} = M$$

$w\frac{\delta x^2}{2}$ is very small.

$$\therefore \quad \delta M = F\delta x$$

$$\text{or} \quad \frac{dM}{dx} = F.$$

The stress at any point in the plane CD distant y from O — O, the neutral plane, will be $f = \frac{My}{I}$.

Let the equilibrium of the area ABCD be considered. On any strip on the section AC having the breadth b, the longitudinal thrust will be

$$f.b.dy = \frac{M.y.b.dy}{I}.$$

Also on any similar strip on BD the longitudinal thrust will be

$$f.b.dy = \frac{(M + \delta M)\, y.b.dy}{I}$$

and the excess of thrust on the strip BD over that on the similar strip on AC

$$= \frac{\delta M.y.b.dy}{I}.$$

And the total excess of thrust on the area BB DD over that on the area AA CC

$$= \frac{\delta M}{I}\int_{y}^{y_1} y.b.dy.$$

where y_1 is the distance of A from the NA.

Then as the thrust on BD exceeds the thrust on AC, and as the area ABCD is in equilibrium, the result must be due to the shearing resistance along the plane CD.

Let q = the intensity of shear stress on the plane CD b at the distance y from the NA, the statement for q will be

$$q.b.\delta x = \frac{\delta M}{I}\int_{y}^{y_1} y.b.dy.$$

$$\text{and } q = \frac{\delta M}{I.b.\delta x}\int_{y}^{y_1} y.b.dy.$$

Substituting F for $\frac{\delta M}{\delta x}$ (see above), the total shearing on the section, the statement for q becomes

$$q = \frac{F}{Ib}\int_{y}^{y_1} y.b.dy.$$

The statement $\int_{y}^{y_1} y.b.dy$ equals the first moment of the area between the limits of y and y_1 about the NA. The ordinates to the distribution curves of any section, at any line distant y from the NA, equals the summation of the moments of the elements of area between the limits of y and y_1. The three following cases of a rectangular section, an I beam and an L, the two latter taken with square angles for the sake of clearness, are illustrated on p. 483, chapter on Statics.

EXAMPLE.—Determine the values for the intensity and distribution curves for a rectangular section 12″ × 6″ (Fig. 368).

$$q = \frac{F}{Ib}\int_{y}^{y_1} y\, b\, dy$$

$$= \frac{F}{I}\left[\frac{y_1^2 - y^2}{2}\right]$$

Substitute for y_1 and y to get the intensity values. $y_1 = \frac{12}{2} = 6$

At NA	$y = 0$	$q = \frac{F}{I}\,18$
At 2″ above the NA,	$y = 2$	$q = \frac{F}{I}\,16$
At 4″ above the NA,	$y = 4$	$q = \frac{F}{I}\,10$
At 6″ above the NA,	$y = 6$	$q = 0$

To get the distribution curve values multiply the intensity values by the breadth of the section.

Thus	At the NA	$\frac{F}{I}\ 18 \times 6 = \frac{F}{I}\ 108$
	At 2″ above the NA	$\frac{F}{I}\ 16 \times 6 = \frac{F}{I}\ 96$
	At 4″ above the NA	$\frac{F}{I}\ 10 \times 6 = \frac{F}{I}\ 60$

The distribution and intensity curves may be drawn from the resistance area, or area of uniform intensity of stress, shown by the shaded triangles on Fig. 368. The latter may be taken to represent the difference in the direct stresses between two transverse sections, and therefore the area of that portion of the triangle above any line parallel to the NA may be taken to represent the shearing stress at that level. These areas, drawn as ordinates to scale, will show the intensity curve, and multiplied by the breadth of the section, will represent the distribution of the shearing at that point. The ordinates to the intensity curve multiplied by $\frac{F}{I}$ will give the actual values of q. The ordinates to the distribution curve multiplied by $\frac{F}{Ib}$ will also give the actual value of q.

EXAMPLE.—Determine the intensity and distribution curves for a 10″ × 8″ I section similar to an R.S.J. Let the angles be taken square and the flanges parallel (Fig. 370).

Assume	Thickness of web	$= b_w = 0{\cdot}6$ in.
	Thickness of flange	$= b_f = 0{\cdot}97$ in.
	Moment of inertia	$= I = 345$ in.^{4}
	Reaction = shear force	$= F = 22{\cdot}8$ tons.

In the equation given above

$$q = \frac{F}{Ib}\int_{y}^{y_1} y.b.dy$$

the values of y and b following the integration sign refer to corresponding and varying values of y and b between the limits of integration y_1 and y. The value of b outside the integration sign and y, the lower limit of integration are the values of b and y corresponding to the height for which q is being calculated.

Thus in this example the second b, which is variable, will have two values, namely b_w and b_f. The statement for this case then becomes

$$q = \frac{F}{Ib_w}\left\{\int_{4\cdot03}^{5} y.b_f.dy + \int_{y}^{4\cdot03} y\,b_w\,dy\right\}$$

$$= \frac{F}{Ib_w}\left\{b_f\int_{4\cdot03}^{5} y\,dy + b_w\int_{y}^{4\cdot03} y\,dy\right\}$$

$$= \frac{22\cdot8}{345\times 0\cdot6}\left\{8\left(\frac{5^2-4\cdot03^2}{2}\right) + 0\cdot6\left(\frac{4\cdot03^2-y^2}{2}\right)\right\}$$

$$q = 4\cdot39 - 0\cdot03\,y^2.$$

At the NA $y = 0$ and $q = 4\cdot39$ tons/in.^{2}

1″ from NA $q = 4\cdot39 - 0\cdot03 \times 1^2 = 4\cdot36$ tons/in.^{2}

2″ ,, ,, $q = 4\cdot39 - 0\cdot03 \times 2^2 = 4\cdot26$,,

3″ ,, ,, $q = 4\cdot39 - 0\cdot03 \times 3^2 = 4\cdot09$,,

4·03″ ,, ,, $q = 4\cdot39 - 0\cdot03 \times 4\cdot03^2 = 3\cdot85$,,

At an infinitely small distance from 4·03 the width changes from ·6 to 8 inches.

$$= \frac{8''}{\cdot6} = 13\cdot3. \quad \text{Then } q = \frac{3\cdot85}{13\cdot3} = 0\cdot29 \text{ tons/in.}^2$$

at 5 inches from NA $q = 0$.

To determine the ordinates for the distribution curve multiply the value of q by the breadth of the section. Thus:

at the NA $O_d = q \times 0\cdot6$

$= 4\cdot39 \times 0\cdot6 = 2\cdot63$ tons.

at 1″ from NA $O_d = 4\cdot36 \times 0\cdot6 = 2\cdot61$,,

2″ ,, ,, $= 4\cdot26 \times \cdot6 = 2\cdot55$,,

3″ ,, ,, $= 4\cdot09 \times \cdot6 = 2\cdot45$,,

4·03″ ,, ,, $= 3\cdot85 \times \cdot6 = 2\cdot31$,,

The effect of the horizontal and vertical shearing stresses is to cause buckling in the web. To determine the safe shearing stress, consider the stress to be acting through an angle of 45 degrees through the web, then find by the formula for pillars the safe value for p_c; this should be the maximum allowable shear for that depth and thickness of web.

Thus depth of web = 9·06″

Length of diagonal = $9{\cdot}06 \times \sqrt{2} = 12{\cdot}77''$

value of r for ·6 = $\sqrt{\frac{{\cdot}6^2}{12}} = {\cdot}17$

$$\frac{l}{r} = \frac{12{\cdot}77}{{\cdot}17} = 75$$

value of p_c for $\frac{l}{r} = 75$ (Rankine) = 5·9 tons/in.²

∴ When $q = 4{\cdot}39$, web is safe from buckling.

EXAMPLE.—Determine the values for the intensity and distribution curves for a 4″ × 4″ × ½″ angle (Fig. 371), the NA passing through the CG parallel to and 1·17 inches from one of the sides.

Take the upper side :

$$q = \frac{F}{Ib}\int_y^{y_1} y\,b\,dy$$

$$= \frac{F}{I}\left[\frac{y_1^2 - y^2}{2}\right]$$

Substitute for y_1 and y for the intensity values.

$y_1 = 4 - 1{\cdot}17 = 2{\cdot}83$ inches.

At the NA, $y = 0$ $\quad q = \frac{F}{I}\,4{\cdot}01$

At 1″ above the NA, $y = 1$ $\quad q = \frac{F}{I}\,3{\cdot}51$

At 2″ above the NA, $y = 2$ $\quad q = \frac{F}{I}\,2{\cdot}01$

For the part below the NA :

$$q = \frac{F}{Ib_2}\left\{\int_{0{\cdot}67}^{1{\cdot}17} y\,b_1\,dy + \int_y^{0{\cdot}67} y\,b_2\,dy\right\}$$

$$q = \frac{F}{I \times 0{\cdot}5}\left\{4\left(\frac{1{\cdot}17^2 - 0{\cdot}67^2}{2}\right) + 0{\cdot}5\left(\frac{0{\cdot}67^2 - y^2}{2}\right)\right\}$$

$$= \frac{F}{I}\,(3{\cdot}90 - 0{\cdot}5\,y^2)$$

Intensity value :

At 0·67″ below the NA $q = \frac{F}{I}\,(3{\cdot}90 - 0{\cdot}5 \times 0{\cdot}67^2)$

$$= \frac{F}{I}\,3{\cdot}68$$

At an infinitely small distance below 0·67 inches from the NA the width suddenly increases to 4 inches.

Ratio of widths equals $\frac{4}{0{\cdot}5} = 8$

$$\therefore\ q = \frac{1}{8}\left\{\frac{F}{I}3{\cdot}68\right\} = \frac{F}{I}0{\cdot}46$$

The distribution curve values are obtained by multiplying the intensity values by the breadth of the section at the particular value taken for y. Thus

At 0·67″ below the NA	$\frac{F}{I}3{\cdot}68 \times 0{\cdot}5 = \frac{F}{I}1{\cdot}84$
At the NA	$\frac{F}{I}4{\cdot}01 \times 0{\cdot}5 = \frac{F}{I}2{\cdot}01$
At 1″ above the NA	$\frac{F}{I}3{\cdot}51 \times 0{\cdot}5 = \frac{F}{I}1{\cdot}71$
At 2″ above the NA	$\frac{F}{I}2{\cdot}01 \times 0{\cdot}5 = \frac{F}{I}1{\cdot}01$

Bending Moment, Curvature, Slope, Deflection.—Let the

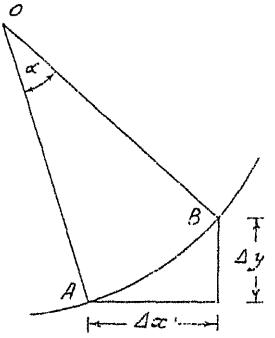


Fig. 421.

given Fig. 421 be a very flat curve and (Δx, Δy) the co-ordinates of point B. Then as the angle α is small, the arc AB and the co-ordinate Δx are nearly equal, and the change in $\frac{dy}{dx}$ corresponds practically to a change in angle (tan θ $=\theta$ where θ is very small). Now the change in

direction or curvature between A and B = the change in $\frac{dy}{dx}$ and the $\left(\text{change in } \frac{dy}{dx}\right) \div$ AB = the average curvature between A and B. If α becomes indefinitely small $\frac{dy}{dx} \div \Delta x$ becomes the rate of change of $\frac{dy}{dx}$ with regard to x; i.e., $\frac{d}{dx}\left(\frac{dy}{dx}\right) = \frac{d^2y}{dx^2} = \frac{1}{R}$ = the curvature of the curve at any point. It is approximately correct and may also be assumed for cases where the curvature is variable.

The curvature or bending produced in a beam is directly related to the intensity of stress to which it is subjected, while the stress is within the elastic limit. Curvature is the change of direction per unit length of curve. A circle has constant curvature, and equals the angle divided by the distance passed through. Thus the angle made in turning through a complete circle is 2π, and the distance is $2\pi r$, then $\frac{2\pi}{2\pi r} = \frac{1}{r}$. The curvature of other plane curves is variable, and is given as the reciprocal of the radius of the circle that most closely agrees with the given curve at any point.

If a beam be subjected to a constant stress throughout its length, it bends to the arc of a circle with a radius R.

It has been shown that for a beam,

$$\frac{1}{R} = \frac{M}{EI}$$

$$\therefore \quad \frac{d^2y}{dx^2} = \frac{M}{EI} \quad \text{or} \quad EI\frac{d^2y}{dx^2} = M$$

If we integrate this expression we get values for $\frac{dy}{dx}$ and for y for any point distance x along the beam. Knowing the bending moment M, these quantities can be evaluated.

Now

$\frac{dy}{dx}$ = slope of beam at any point.

y = deflection of beam at any point.

Thus

$$EI\frac{d^2y}{dx^2} = M$$

$$EI\frac{dy}{dx} = \int M \,.\, dx$$

$$EIy = \iint M\, dx \,.\, dx$$

On pp. 556/7 there are these two relations :—

$$F = \frac{dM}{dx} \qquad w = \frac{dF}{dx}.$$

Then again $$EI\frac{d^2y}{dx^2} = M.$$

Differentiating $$EI\frac{d^3y}{dx^3} = \frac{dM}{dx} = F$$

$$\therefore \quad EI\frac{d^3y}{dx^3} = F.$$

Differentiating again we have

$$EI\frac{d^4y}{dx^4} = \frac{dF}{dx} = w$$

$$\therefore \quad EI\frac{d^4y}{dx^4} = w$$

where F = shear force at any point
and w = load per unit length.

Note these two relations with regard to the examples on deflection in the chapter on Graphic Statics.

$$w = \frac{d^2M}{dx^2}. \text{ and } \frac{M}{EI} = \frac{d^2y}{dx^2}$$

Deflection.—In permanent structures it is required that a beam, in addition to being sufficiently strong, shall be stiff enough to prevent undue bending.

Then in a straight beam let : —

x = distance measured from known point on beam
y = deflection at distance x
M = bending moment at distance x
E = modulus of elasticity
I = moment of inertia of section.

The deflection can now be found.

Thus taking the standard cases previously given for determining the moment of the load M :—

Case 1.—Cantilever, loaded at free end.

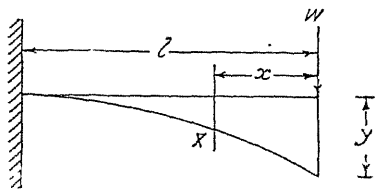


Fig. 422.

Now $$M = EI\frac{d^2y}{dx^2}$$

Write down general expression for bending moment at any section x

$$EI\frac{d^2y}{dx^2} = Wx \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (1)$$

$$EI\frac{dy}{dx} = \frac{Wx^2}{2} + C \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (2)$$

$$EIy \quad = \frac{Wx^3}{6} + Cx + D \quad . \quad . \quad . \quad . \quad . \quad (3)$$

Slope or $\frac{dy}{dx} = 0$ when $x = l$.

Substitute in (2)

$$0 = \frac{Wl^2}{2} + C$$

$$\therefore \quad C = -\frac{Wl^2}{2}.$$

Deflection or $y = 0$ when $x = l$.

Substitute in (3)

$$0 = \frac{Wl^3}{6} - \frac{Wl^2}{2} \cdot l + D$$

$$D = \frac{Wl^3}{2} - \frac{Wl^3}{6}$$

$$D = \frac{Wl^3}{3}.$$

Equation (3) becomes

$$EIy = \frac{Wx^3}{6} - \frac{Wl^2}{2} \cdot x + \frac{Wl^3}{3}.$$

Deflection is a maximum where $x = 0$.

$$\therefore \quad EIy = \frac{Wl^3}{3}$$

$$y = \frac{Wl^3}{3EI}.$$

Case 2.—Cantilever under a distributed load.

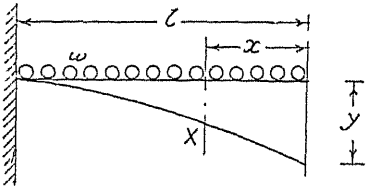


Fig. 423.

Write general expression for bending moment at section X, Fig. 423.

$$EI\frac{d^2y}{dx^2} = \frac{wx^2}{2} \quad \ldots\ldots\ldots\ldots \quad (1)$$

$$EI\frac{dy}{dx} = \frac{wx^3}{6} + C. \quad \ldots\ldots\ldots\ldots \quad (2)$$

$$EIy = \frac{wx^4}{24} + Cx + D \quad \ldots\ldots\ldots\ldots \quad (3)$$

Slope = 0 where $x = l$.

Substitute in (2)

$$0 = \frac{wl^3}{6} + C$$

$$\therefore \quad C = -\frac{wl^3}{6}.$$

Deflection = 0 when $x = l$.

Substitute in (3)

$$0 = \frac{wl^4}{24} - \frac{wl^3}{6} \cdot l + D$$

$$D = \frac{wl^4}{6} - \frac{wl^4}{24}$$

$$D = \frac{wl^4}{8}.$$

Equation (3) becomes

$$EIy = -\frac{wx^4}{24} - \frac{wl^3}{6} \cdot x + \frac{wl^4}{8}.$$

Deflection is a maximum where $x = 0$.

$$\therefore \quad EIy = \frac{wl^4}{8}. \qquad \text{Now } W = wl$$

$$\therefore \quad y = \frac{Wl^3}{8EI}.$$

Case 3.—Beam supported at ends, centrally loaded Fig. 424.

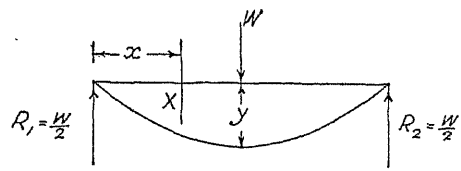


Fig. 424.

Write general expression for bending moment at section X

$$EI\frac{d^2y}{dx^2} = R_1 x \quad \ldots\ldots\ldots\ldots \quad (1)$$

$$EI\frac{dy}{dx} = R_1\frac{x^2}{2} + C \quad \ldots\ldots\ldots \quad (2)$$

$$EIy = R_1\frac{x^3}{6} + Cx + D \quad \ldots\ldots \quad (3)$$

Deflection = 0 when $x = 0$.

Substitute in (3). $\therefore D = 0$.

Slope = 0 when $x = \frac{l}{2}$.

Substitute in (2)

$$0 = R_1\frac{l^2}{8} + C.$$

$$\therefore \quad C = -\frac{W}{2} \cdot \frac{l^2}{8} = -\frac{Wl^2}{16}.$$

Equation (3) becomes

$$EIy = \frac{W}{2} \cdot \frac{x^3}{6} - \frac{Wl^2}{16} \cdot x$$

$$= \frac{Wx^3}{12} - \frac{Wl^2}{16} \cdot x.$$

Maximum deflection occurs when $x = \frac{l}{2}$.

$$\therefore \quad EIy = \frac{Wl^3}{96} - \frac{Wl^3}{32} = -\frac{Wl^3}{48}$$

$$y = \frac{Wl^3}{48EI} \text{ (neglecting negative sign).}$$

Case 4.—Beam supported at ends, load uniformly distributed. Fig. 425.

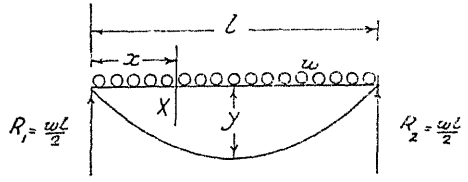


Fig. 425.

Write general expression for bending moment at section X

$$EI\frac{d^2y}{dx^2} = R_1x - \frac{wx^2}{2} \quad \ldots\ldots\ldots \quad (1)$$

$$EI\frac{dy}{dx} = R_1\frac{x^2}{2} - \frac{wx^3}{6} + C \quad \ldots\ldots \quad (2)$$

$$EIy = R_1\frac{x^3}{6} - \frac{wx^4}{24} + Cx + D \quad \ldots \quad (3)$$

Deflection = 0 when $x = 0$. $\therefore$ D = 0.

Slope = 0 when $x = \frac{l}{2}$ and $R_1 = \frac{wl}{2}$.

Substitute in (2)

$$0 = \frac{wl}{2}\cdot\frac{l^2}{8} - \frac{wl^3}{48} + C$$

$$C = \frac{wl^3}{48} - \frac{wl^3}{16}$$

$$\therefore \quad C = \frac{wl^3}{24}.$$

Equation (3) becomes

$$EIy = \frac{wl}{2}\cdot\frac{x^3}{6} - \frac{wx^4}{24} - \frac{wl^3}{24}\cdot x.$$

Maximum deflection occurs when $x = \frac{l}{2}$.

Substituting

$$EIy = \frac{wl^4}{96} - \frac{wl^4}{384} - \frac{wl^4}{48} = -\frac{5wl^4}{384}$$

$$\therefore \quad y = \frac{5wl^4}{384EI} = \frac{5Wl^3}{384EI} \text{ (neglecting negative sign).}$$

Case 5.—Beam ends fixed, centrally loaded. Fig. 426.

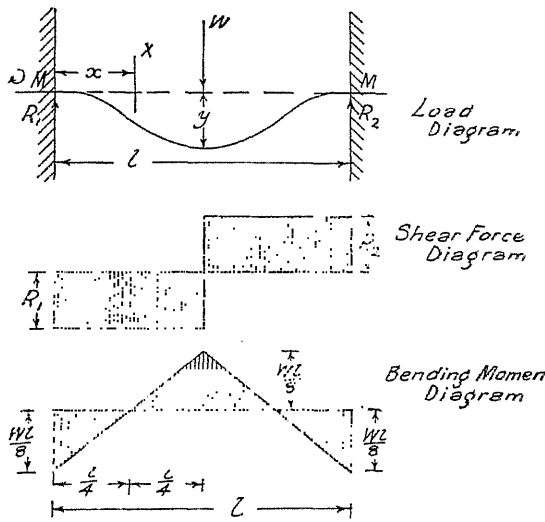


Fig. 426.

In this and the following case it is necessary to determine the bending moment at the centre and the supports in addition to the deflection.

Write general expression for bending moment at section X.

$$EI\frac{d^2y}{dx^2} = M - R_1x \quad \ldots\ldots\ldots \quad (1)$$

$$EI\frac{dy}{dx} = Mx - R_1\frac{x^2}{2} + C \quad \ldots\ldots \quad (2)$$

$$EIy = M\frac{x^2}{2} = R_1\frac{x^3}{6} + Cx + D \quad \ldots \quad (3)$$

Deflection = o when x = o. $\therefore$ D = o.
Slope = o when x = o. $\therefore$ C = o.

Slope $= 0$ when $x = \frac{l}{2}$ and $R_1 = \frac{W}{2}$.

Substitute in (2)

$$0 = M \cdot \frac{l}{2} - \frac{W}{2} \cdot \frac{l^2}{8}$$

$$M \cdot \frac{l}{2} = \frac{Wl^2}{16}$$

$$M \text{ (at supports)} = \frac{Wl}{8}.$$

This is the value of the bending moment at the fixed ends. For the bending moment at the centre substitute $x = \frac{l}{2}$ in equation (1).

$$\text{BM (at centre)} = M - R_1 \frac{l}{2}$$

$$= \frac{Wl}{8} - \frac{Wl}{4}$$

$$\text{BM (at centre)} = \frac{Wl}{8}.$$

The maximum deflection at the centre can now be calculated.

Substitute $x = \frac{l}{2}$ in equation (3).

$$EIy = M \cdot \frac{x^2}{2} - R_1 \frac{x^3}{6}$$

$$= \frac{Wl}{8} \cdot \frac{l^2}{8} - \frac{W}{2} \cdot \frac{l^3}{48}$$

$$= \frac{Wl^3}{64} - \frac{Wl^3}{96} = \frac{Wl^3}{192}$$

$$\therefore \quad y \text{ (max.)} = \frac{Wl^3}{192\,EI}.$$

The bending moments at centre and at the supports are equal in amount but opposite in sign. The point at which the change over occurs is known as the point of contraflexure.

For point of contraflexure equate bending moment expression (1) to zero.

$$M - R_1 x = 0$$

$$\frac{Wl}{8} - \frac{W}{2} \cdot x = 0$$

$$\frac{Wx}{2} = \frac{Wl}{8}$$

$$x = \frac{l}{4}.$$

Thus the points of contraflexure are $\frac{l}{4}$ from the ends of the beam.

Case 6.—Beam. Ends fixed, load distributed. Fig. 427.

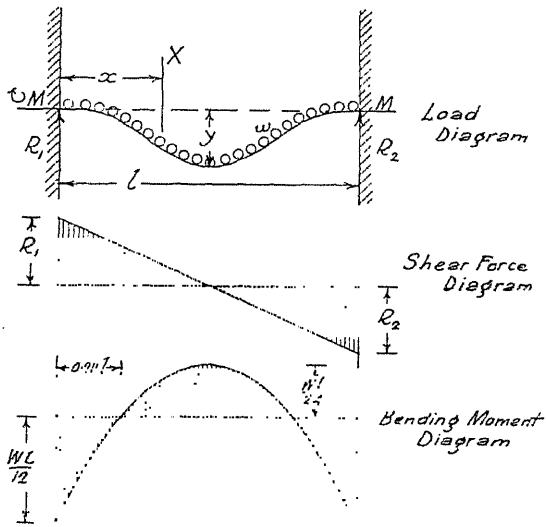


Fig. 427.

Write general expression for bending moment at section X.

$$EI\frac{d^2y}{dx^2} = M - R_1 x + \frac{wx^2}{2} \quad \ldots\ldots\ldots \quad (1)$$

$$EI\frac{dy}{dx} = Mx - R_1\frac{x^2}{2} + \frac{wx^3}{6} + C \quad \ldots\ldots \quad (2)$$

$$EIy = M\frac{x^2}{2} - R_1\frac{x^3}{6} + \frac{wx^4}{24} + Cx + D \quad . \quad . \quad (3)$$

Deflection $= 0$ when $x = 0$. $\therefore \quad D = 0.$
Slope $= 0$ when $x = 0$. $\therefore \quad C = 0.$

Slope $= 0$ when $x = \frac{l}{2}$

Substitute in (2) $x = \frac{l}{2}$ $\qquad R_1 = \frac{wl}{2}$

$$0 = M \cdot \frac{l}{2} - \frac{wl}{2} \cdot \frac{l^2}{8} + \frac{wl^3}{48}$$

$$M \cdot \frac{l}{2} = \frac{wl^3}{16} - \frac{wl^3}{48} = \frac{wl^3}{24}$$

$\therefore$ M (at supports) $= \frac{wl^2}{12} = \frac{Wl}{12}$.

For the bending moment at the centre substitute values obtained in equation (1), $x = \frac{l}{2}$.

$$\text{BM (at centre)} = \frac{Wl}{12} - \frac{W}{2} \cdot \frac{l}{2} + \frac{wl^2}{8}$$

$$= \frac{Wl}{12} - \frac{Wl}{4} + \frac{Wl}{8}$$

$$\text{BM (at centre)} = \frac{Wl}{24}$$

The maximum deflection is at the centre. Substitute $x = \frac{l}{2}$ in equation (3).

$$EIy = M\frac{x^2}{2} - R_1\frac{x^3}{6} + \frac{wx^4}{24}$$

$$= \frac{wl^2}{12} \cdot \frac{l^2}{8} - \frac{wl}{2} \cdot \frac{l^3}{48} + \frac{wl^4}{384}$$

$$= \frac{wl^4}{96} - \frac{wl^4}{96} + \frac{wl^4}{384} \qquad = \frac{wl^4}{384}$$

$$\therefore \quad y = \frac{Wl^3}{384EI}$$

The maximum bending moment is at the end. For the point of contraflexure equate equation (1) to zero.

$$M - R_1 x + \frac{wx^2}{2} \quad = 0$$

$$\frac{Wl}{12} - \frac{W}{2} \cdot x + \frac{wx^2}{2} = 0$$

or

$$\frac{wl^2}{12} - \frac{wlx}{2} + \frac{wx^2}{2} = 0$$

$$\frac{l^2}{6} - lx + x^2 = 0$$

$$x^2 - lx + \frac{l^2}{6} = 0.$$

Solving

$$x = \frac{1}{2}\left\{ l \pm \sqrt{l^2 - \frac{4l^2}{6}} \right\}$$

$$= \frac{1}{2}\left\{ l \pm \sqrt{\frac{l^2}{3}} \right\}$$

$$= \frac{1}{2}\left(l \pm \frac{l}{\sqrt{3}} \right)$$

$$= \frac{l}{2}\left(1 \pm \frac{1}{\sqrt{3}} \right)$$

$$= 0{\cdot}211l \text{ or } 0{\cdot}788l.$$

Case 7.—Gallery beam (Fig. 428). The beam is to be 20 feet long, fixed in the wall and supported at a distance of 8 feet from the wall. The weight of the floor is carried on cross girders which fall on the beam in the position shown and with the loads indicated in the diagram.

Let M = moment at fixed end.

It is required to find the values of the reactions R_1 and R_2, and also the bending moment and deflection at any point on the beam.

Obtain expressions for the bending moment at four points, *i.e.*, between

R_1 and R_2
R_2 ,, W_1
W_1 ,, W_2
W_2 ,, W_3

$$M_x = EI\frac{d^2y}{dx^2}. \quad \text{Measure } x \text{ from } R_1$$

(1) $EI\frac{d^2y}{dx^2}$	$= M + R_1x$	$-R_2(x-a)$	$+W_1(x-b)$	$+W_2(x-c)$
(2) $EI\frac{dy}{dx}$	$= Mx + \frac{R_1x^2}{2} + C$	$-\frac{R_2(x-a)^2}{2}$	$+\frac{W_1(x-b)^2}{2}$	$+\frac{W_2(x-c)^2}{2}$
(3) EIy	$= \frac{Mx^2}{2} + \frac{R_1x^3}{6} + Cx + D$	$-\frac{R_2(x-a)^3}{6}$	$+\frac{W_1(x-b)^3}{6}$	$+\frac{W_2(x-c)^3}{6}$

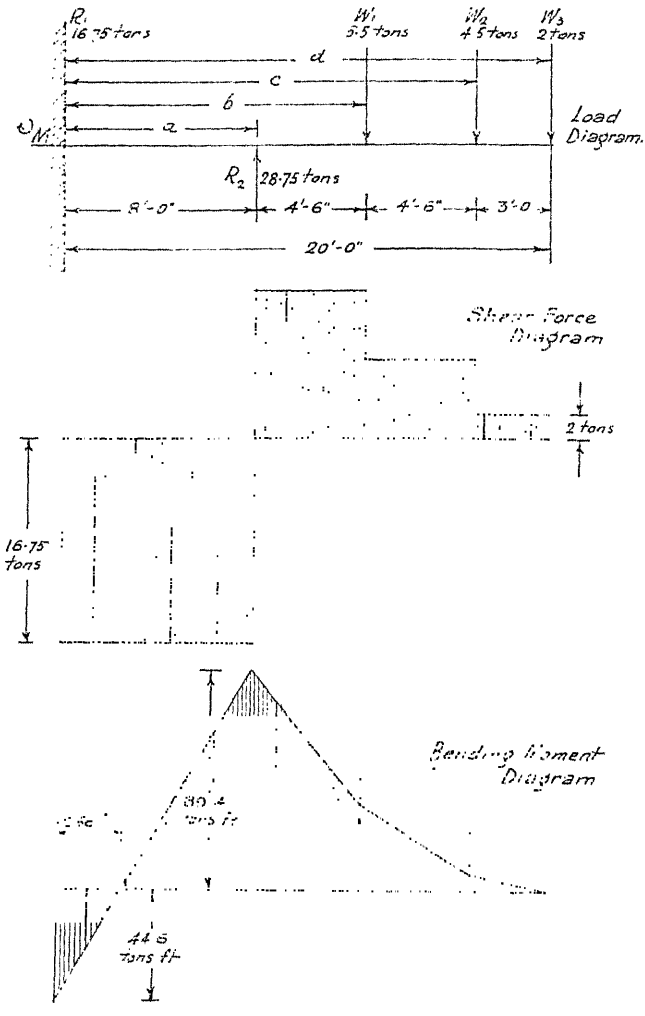

R1
16·75 tons
W1
5·5 tons
W2
4·5 tons
W3
2 tons
d
c
b
a
Load Diagram.
M
R2 28·75 tons
8'-0"
4'-6"
4'-6"
3'-0
20'-0"
Shear Force Diagram
2 tons
16·75 tons
Bending Moment Diagram
44·5 tons ft.

Fig. 428.

Consider the conditions which can be used to determine the unknowns M, R_1, R_2, C and D.

$$\frac{d^2y}{dx^2} = \text{o when } x =$$

$$\frac{dy}{dx} = \text{o when } x = \text{o} \quad \therefore \quad C = \text{o}$$

$$y = \text{o when } x = \text{o} \quad \therefore \quad D = \text{o}$$

$$y = \text{o when } x = a$$

$$R_1 + W_1 + W_2 + W_3 = R_2.$$

Substitute $x = l$ in equation (1). As the end of the beam is being considered the whole of equation (1) must be used.

Also substitute $x = a$ in equation (3). As this point is in the first section of the beam only the first section of the equation (3) is used.

$$= M + R_1 l - R_2(l - a) + W_1(l - b) + W_2(l - c) \quad . \quad . \quad . \quad (4)$$

$$\text{o} = \frac{Ma^2}{2} + \frac{R_1 a^3}{6} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (5)$$

Multiply (4) by $\frac{a^2}{2}$ and solve R_1. We get

$$\text{o} = M\frac{a^2}{2} + R_1 l\frac{a^2}{2} - R_2(l - a)\frac{a^2}{2} + W_1(l - b)\frac{a^2}{2} + W_2(l - c)\frac{a^2}{2}$$

$$\text{o} = M\frac{a^2}{2} + R_1\frac{a^3}{6}.$$

Subtracting

$$\text{o} = R_1\left(\frac{la^2}{2} - \frac{a^3}{6}\right) - R_2(l - a)\frac{a^2}{2} + W_1(l - b)\frac{a^2}{2} + W_2(l - c)\frac{a^2}{2}$$

$$R_2 = R_1 + W = R_1 + 12$$

$$R_1\left\{\frac{20 \times 64}{2} - \frac{512}{6}\right\} = (R_1 + 12)\left\{(20 - 8)\frac{64}{2}\right\} + 5{\cdot}5(20 - 12{\cdot}5)\frac{64}{2}$$

$$+ 4{\cdot}5\,(20 - 17)\frac{64}{2}$$

$$\therefore \quad R_1 = 16{\cdot}75 \text{ tons.}$$

Substitute in (5).

$$\text{o} = \frac{M64}{2} + \frac{16{\cdot}75 \times 512}{6}$$

$$\therefore \quad M = -44{\cdot}6 \text{ tons ft.}$$

Substitute for M and R_1 in (4),

$$0 = -44{\cdot}6 + 16{\cdot}75 \times 20 - R_2(20-8) + 5{\cdot}5(20-12{\cdot}5) + 4{\cdot}5(20-17)$$

$$\therefore\ R_2 = +28{\cdot}75 \text{ tons.}$$

Now draw the bending moment diagram. Obtain various values for the bending moment by substituting values for x in (1), the bending moment equation.

When $x = 2$ $M_x = -44{\cdot}6 + 16{\cdot}75 \times 2 = -11{\cdot}1$ tons ft.
$x = 4$ $= -44{\cdot}6 + 16{\cdot}75 \times 4 = 22{\cdot}4$,,
$x = 6$ $= -44{\cdot}6 + 16{\cdot}75 \times 6 = 55{\cdot}9$,,
$x = 8$ $= -44{\cdot}6 + 16{\cdot}75 \times 8 = 89{\cdot}4$,,
$x = 12{\cdot}5$ $= -44{\cdot}6 + 16{\cdot}75 \times 12{\cdot}5 - 28{\cdot}75 \times 4{\cdot}5 = 35$ tons ft.
$x = 17$ $= -44{\cdot}6 + 16{\cdot}75 \times 17 - 28{\cdot}75 \times 9 + 5{\cdot}5 \times 4{\cdot}5$ $= 6{\cdot}15$ tons ft.

It will be seen that the maximum bending moment occurs over the support and equals 89·4 tons feet.

To calculate beam required.

$$\frac{M}{f} = Z \quad \text{Let } f = 8 \text{ tons/square inch}$$

$$Z = \frac{89{\cdot}4 \times 12}{8} = 134 \text{ inches}^3.$$

Take as a suitable beam $22'' \times 7'' \times 75$ lbs.

Test for deflection. The deflection at end must not be more than $\frac{l}{400} = 0{\cdot}60$ inches.

Substitute in (3).

$$EIy = \frac{44{\cdot}6 \times 400}{2} + \frac{16{\cdot}75 \times 8000}{6} - \frac{28{\cdot}75 \times 12^3}{6} + \frac{5{\cdot}5 \times 7{\cdot}5^3}{6} + \frac{4{\cdot}5 \times 3^3}{6}$$

$$= 5486 \text{ tons ft.}^3$$

$$\therefore\ y = \frac{5486 \times 12^3}{13000 \times 1677}$$

$$= 0{\cdot}44 \text{ inch.}$$

Hence the beam is safe in deflection.

There is a point of contraflexure between R_1 and R_2 where the bending moment is zero. Substitute in first portion of (1).

$$M + R_1 x = 0$$

$$-44{\cdot}6 + 16{\cdot}75x = 0$$

$$\therefore\ x = \frac{44{\cdot}6}{16{\cdot}75}$$

$$x = 2{\cdot}66 \text{ feet.}$$

$\therefore$ Point of contraflexure is at 2·66 feet from the wall.

A drawing of the above beam is given in detail in the author's *Elementary Course*. The beam is bent over the

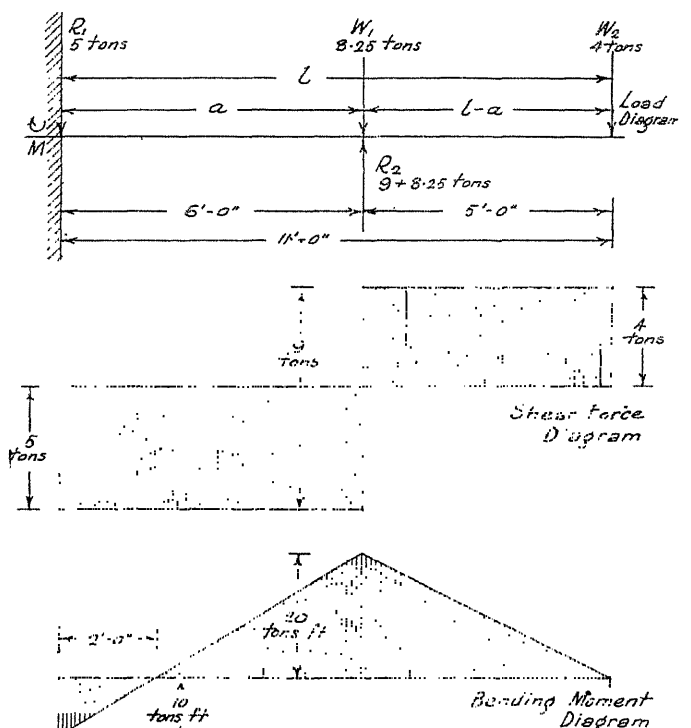


Fig. 429.

support at R_2. Between W_2 and W_3 the section of the beam is changed.

Case 8.—Gallery beam (Fig. 429). The beam is to be 11 feet long, fixed in the wall and supported at a distance of 6 feet from the wall. The loading is 2 cwts. per square foot. This is carried by longitudinal girders which concentrate the load on the beam in the positions shown in the diagram.

The trusses are spaced 15 feet centres. Hence the total load $= 15 \times 11 \times 2 = 330$ cwts.

Distribution of load.

$$\text{Load on wall bay} = \frac{15 \times 6 \times 2}{20} = 9 \text{ tons.}$$

$$\text{Load on outer bay} = \frac{15 \times 5 \times 2}{20} = 7{\cdot}5 \text{ tons.}$$

Hence

Load on wall $= 4{\cdot}5$ tons.
Load on centre post $= 4{\cdot}5 + 3{\cdot}75 = 8{\cdot}25$ tons.
Load on end $= 3{\cdot}75$ tons, say 4 tons.

It is necessary to find the reactions, and the bending moment at any point on the beam.

Obtain general expressions for the bending moment in the two sections of the beam, *i.e.*, between the wall and the support and between the support and the end.

$$Mx = EI\frac{d^2y}{dx^2}. \quad \text{Measure } x \text{ from wall.}$$

$$EI\frac{d^2y}{dx^2} = M + R_1x \qquad - R_2(x - a) \quad . \quad . \quad . \quad . \quad (1)$$

$$EI\frac{dy}{dx} = Mx + \frac{R_1x^2}{2} + C \qquad - R_2\frac{(x-a)^2}{2} \quad . \quad . \quad . \quad . \quad (2)$$

$$EIy = \frac{Mx_2}{2} + \frac{R_1x^3}{6} + Cx + D \qquad - R_2\frac{(x-a)^3}{6} \quad . \quad . \quad . \quad . \quad (3)$$

Consider the known conditions

$$\frac{d^2y}{dx^2} = 0 \text{ when } x = l$$

$$\frac{dy}{dx} = 0 \text{ when } x = 0 \quad \therefore C = 0$$

$$y = 0 \text{ when } \left.\begin{matrix} x = 0 \\ x = a \end{matrix}\right\} \quad \therefore D = 0$$

$$R_1 + W_2 = R_2.$$

Neglect W_1 since it is carried directly by the post. To find M, R_1 and R_2.

Substitute $x = l$ in equation (1). $\therefore \frac{d^2y}{dx^2} \cdot EI = 0$

Substitute $x = a$ in equation (3). $\therefore y = 0$

$$0 = M + R_1 l - R_2(l - a) \quad \ldots\ldots\ldots \quad (4)$$

$$0 = \frac{Ma^2}{2} + \frac{R_1 a^3}{6} \quad \ldots\ldots\ldots\ldots \quad (5)$$

Solving

$$0 = \frac{Ma^2}{2} + \frac{R_1 l a^2}{2} - \frac{R_2(l - a)a^2}{2}$$

$$0 = \frac{Ma^2}{2} + \frac{R_1 a^3}{6}.$$

Subtracting

$$0 = R_1\left(\frac{la^2}{2} - \frac{a^3}{6}\right) - R_2(l - a)\frac{a^2}{2}$$

Now $R_2 = R_1 + 4$

$$0 = R_1\left(\frac{11 \times 36}{2} - \frac{216}{6}\right) - \left(\frac{5 \times 36}{2}\right)(R_1 + 4)$$

$\therefore \quad R_1 = 5$ tons.

Substitute in (5)

$$0 = \frac{36M}{2} + \frac{5 \times 216}{6}$$

$\therefore \quad M = -10$ tons ft.

Substitute for M and R_1, in (4)

$$0 = -10 + 5 \times 11 - 5R_2$$

$\therefore \quad R_2 = 9$ tons.

By substituting in (1) as in previous example we obtain values for the bending moment and hence can draw the bending moment curve.

The maximum bending moment occurs over the support and equals 20 tons feet.

To calculate beam required.

$$\frac{M}{f} = Z$$

$$\therefore \quad Z = \frac{20 \times 12}{8} = 30 \text{ inches}^3.$$

Take as a suitable beam 12″ × 5″ × 32 lbs.

Test for deflection. The deflection at the end must not be more than $\frac{l}{400} = 0{\cdot}33$ inch.

Substitute in (3).

$$EIy = \frac{-10 \times 121}{2} + \frac{5 \times 1331}{6} - \frac{9 \times 125}{6}$$

$$\therefore y \text{ (in feet)} = \frac{1}{EI} \cdot 318$$

$$\therefore y \text{ (in inches)} = \frac{318 \times 12 \times 12 \times 12 \times 2240}{29{,}000{,}000 \times 206{\cdot}9}$$

$$= 0{\cdot}20 \text{ inch.}$$

Hence the beam is safe in deflection.

There is a point of contraflexure between R_1 and R_2. Where

$$BM = 0$$
$$M + R_1 x = 0$$
$$-10 + 5x = 0$$
$$x = 2 \text{ feet.}$$

∴ Point of contraflexure is at 2 feet from the wall.

Case 9 (Fig. 430).—This example has been worked out graphically in the chapter on "Graphic Statics," p. 472.

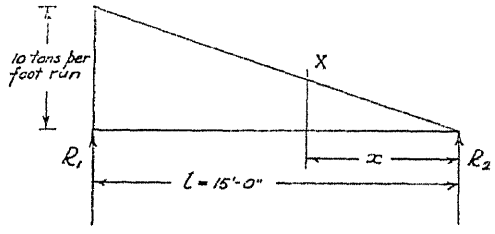


Fig. 430.

A beam is simply supported and carries a uniformly increasing load which varies from zero at the right-hand end to a maximum of 10 tons per foot run at the other.

It is required to calculate the maximum deflection.

If the load at unit distance from $R_2 = w$ tons per foot run, the intensity of loading at $R_1 = wl$ tons per foot run.

$$\therefore \quad wl = 10$$
$$w = 0{\cdot}67 \text{ tons/foot run.}$$
$$\text{Total load} = \frac{wl^2}{2} = \frac{{\cdot}67 \times 15^2}{2}$$
$$W = 75 \text{ tons.}$$

Now obtain a general expression for the bending moment. Measure x from R_2.

$$Mx = EI\frac{d^2y}{dx^2} = R_2x - \frac{wx^3}{6} \quad . \quad . \quad . \quad . \quad . \quad (1)$$

$$EI\frac{dy}{dx} = R_2\frac{x^2}{2} - \frac{x^4}{24} + C \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (2)$$

$$EIy = \frac{R_2x^3}{6} - \frac{wx^5}{120} + Cx + D \quad . \quad . \quad . \quad . \quad (3)$$

Consider known conditions.

When

$$x = 0 \qquad y = 0 \qquad \therefore D = 0$$
$$x = l \qquad y = 0$$

$$R_1 = \frac{2}{3}W\,; \quad R_2 = \frac{1}{3}W.$$

Substitute latter conditions in (3).

$$0 = \frac{Wl^3}{18} - \frac{wl^5}{120} + Cl$$
$$\therefore \quad C = -655.$$

We require to find the point at which maximum bending moment occurs, that is, the maximum value of equation (1). Differentiate and equate to zero.

$$EI\frac{d^2y}{dx^2} = R_2 - \frac{wx^2}{2} = 0$$
$$\therefore \quad R_2 = \frac{wx^2}{2}$$
$$\frac{W}{3} = \frac{0{\cdot}67 \times x^2}{2} \qquad W = 75 \text{ tons.}$$
$$\therefore \quad x = 8{\cdot}66 \text{ feet.}$$

Thus maximum bending moment occurs at a point 8·66 feet from R_2.

Substitute in (1) to find its value.

$$\text{Max. B.M.} = \frac{75 \times 8{\cdot}66}{3} - \frac{0{\cdot}67 \times 8{\cdot}66^3}{6}$$
$$= 143{\cdot}5 \text{ tons ft.}$$

$$\text{Now } Z = \frac{M}{f} = \frac{143{\cdot}5 \times 12}{8} = 215 \text{ inches}^3.$$

Choose a compound beam consisting of 1 No. 18″ × 6″ × 55 lbs. R.S.J. with 1 No. 10″ × $\frac{7}{8}$″ plates on each flange.

Moment of inertia = 2152 inches^4.

Find point of maximum deflection. This occurs where the slope is zero.

$$\therefore EI\frac{dy}{dx} = 0 = \frac{75 \times x^2}{6} - \frac{0{\cdot}67 \times x^4}{24} - 655$$

$$0{\cdot}0278x^4 - 12{\cdot}5x^2 + 655 = 0.$$

$$\therefore x^2 = 60{\cdot}5$$

$$\therefore x = 7{\cdot}78 \text{ feet.}$$

$\therefore$ Maximum deflection occurs 7·78 feet from R_2.

Substitute in (3) for maximum deflection.

$$EIy = \frac{75 \times 471}{18} - \frac{2 \times 28480}{3 \times 120} - 655 \times 7{\cdot}78$$

$$= 3288 \text{ tons ft.}^3$$

$$= 3288 \times 2240 \times 12 \times 12 \times 12 \text{ lbs. in.}^3$$

$$\therefore y = \frac{3288 \times 2240 \times 12 \times 12 \times 12}{29{,}000{,}000 \times 2152}$$

$$y = 0{\cdot}20 \text{ inch.}$$

Compare this value of the deflection with that obtained graphically (p. 475).

Rectangular Sections.—This form is chiefly restricted to beams of timber. The actual strength of timber beams is difficult to determine owing to the great variation in the essential properties, even in different specimens of the same species. Therefore, where timber is used for the principal girders in any structure, great care should be exercised in its selection, and it is advisable to test such members by subjecting them to loads slightly greater than the proposed load, and, by careful measurements of the deflection, to ascertain from the elasticity thus determined if the log is adequate for the intended purpose.

Great care must be exercised in employing many of the published values of f for timber, as many of these are based on the results of tests made on small specimens. As the latter may be practically perfect in their structure they give

far higher results than could be expected from full sized beams.

It can be established experimentally that the strength of beams vary (1) directly as the breadth, (2) inversely as the length, and (3) as the square of the depth. It would therefore follow that the deeper the beam for any given sectional area, the higher the resulting value, but there are practical limitations which render it necessary to make an ample provision for the breadth.

Even for ordinary floor joists, 2 inches in breadth should be the minimum; anything less would be likely to split when nailing the floor boards. The breadth for this purpose is usually made from ·25 to ·33 of the depth, strutting being provided to give lateral support and prevent buckling. For beams of moderate dimensions a breadth of from ·6 to ·75 of the depth is usually sufficient for lateral resistance. For the largest beams the form and the dimensions of the section are usually limited by the most suitable rectangle that can be cut from the logs of the particular timber employed.

Where sufficient strength cannot be obtained from the use of timber alone it may be supplemented by an iron or steel flitch. Whole timbers should always be sawn longitudinally through the centre to expose the heart and facilitate inspection, also to prevent the star shakes that usually develop during the seasoning of uncut logs. The halves or flitches are turned heart side outwards and bolted together. This deep cut does not reduce the strength of the beam. Where a steel flitch is used it is usually placed between the timbers.

In determining the dimensions of a timber beam the moment of the load is equated with the moment of resistance, *i.e.*—

$$M = \frac{fI}{y} = \frac{f\frac{bd^3}{12}}{\frac{d}{2}} = \frac{fbd^2}{6}.$$

The breadth is usually assumed, or it is given in terms of the depth. The dimensions of a beam arrived at in this way may be perfectly satisfactory to support the given load, but the deflection may be too great for many purposes,

such as for floors which support a plaster ceiling. In such cases to prevent unsightly cracks, it is necessary to limit the deflection to a fraction varying between $\frac{1}{360}$ to $\frac{1}{480}$ of the length. The depth can then be obtained from the formula for deflection, *i.e.*—

$$y = \frac{nWl^3}{EI}$$

where n equals the numerical co-efficient derived from the given conditions of loading and end support.

$$y = \frac{nWl^3}{E\frac{bd^3}{12}}$$

$$d = \sqrt[3]{\frac{nWl^3 12}{y\,Eb}}$$

As a rule it is unnecessary to consider a rectangular beam for shearing, there being an abundance of material at the NA where the intensity of shearing is greatest.

Where timber beams are employed the effective sectional area must not be reduced. An allowance in breadth or depth should be made to compensate for any notching, mortising, etc., required for framing the timbers together.

EXAMPLE.—Determine the dimensions for a beam of northern pine, ends supported, to carry a load of 12 tons distributed over a span of 18 feet ; $f = 2\frac{1}{4}$ tons per square inch, working $f = \frac{3}{4}$ ton per square inch. E = 550 tons per square inch. Let $b = d$, then

$$\frac{Wl}{8} = \frac{fbd^2}{6} = \frac{fd^3}{6}$$

$$d^3 = \frac{6Wl}{8f}$$

$$d = \sqrt[3]{\frac{6 \times 12 \times 216}{8 \times \cdot 75}}$$

$$d = 13\cdot7, \text{ say } 14\cdot0.$$

Then the section = 14 × 14.

If it is desired to limit the deflection to say $\frac{1}{480}$ the

length, the required depth may be obtained as follows: let

$$y = \text{deflection} = \frac{18 \times 12}{480} = \frac{9''}{20}.$$

Then
$$y = \frac{9''}{20} = \frac{nWl^3}{EI} = \frac{5 \times 12 \times 216^3}{384 \times 550 \times \frac{bd^3}{12}}$$

and
$$d = \sqrt[3]{\frac{5 \times 12 \times 216 \times 216 \times 216 \times 12 \times 20}{384 \times 550 \times 14 \times 9}}$$
$$= 17{\cdot}6, \text{ say } 18''.$$

The dimensions of the section would then be $18'' \times 14''$.

A log $18'\ 0'' \times 18'' \times 14''$ would be difficult to obtain. Trees of this species from which good logs of this dimension could be cut are scarce, but good balks suitable for such beams up to 14 inches square are common.

Therefore for the above beam and condition as to deflection it would be better to fix the depth at 14 inches and obtain the necessary stiffness by supplementing the breadth, either by adding another timber, or by the use of an iron or steel flitch. Taking the latter case: determine the proportion of the total load the timber would support without exceeding the given deflection. Secondly, determine the thickness of the plate required to take the remainder of the load, and give the same deflection.

Thus
$$y = \frac{9''}{20} = \frac{nWl^3}{EI}$$
$$= \frac{5\ Wl^3 12}{384.E.bd^3}$$

taking b and $d = 14''$

$$W = \frac{9 \times 384 \times 550 \times 14^4}{20 \times 5 \times 216^3 \times 12}$$
$$= 6{\cdot}04, \text{ say } 6 \text{ tons.}$$

This would leave 6 tons to be taken by the steel plate, and taking $f = 10$ tons, $E = 13{,}000$ tons.

Then
$$y = \frac{9''}{20} = \frac{5 \times Wl^3 \times 12}{384 \times E \times b \times d^3}$$

and
$$b = \frac{5 \times 6 \times 216^3 \times 12 \times 20}{384 \times 13000 \times 14^3 \times 9}$$
$$= {\cdot}59, \text{ say } \tfrac{5}{8} \text{ plate.}$$

It is usual to place the steel plate between the two wood flitches, the whole being bolted together by say $\frac{3}{4}$-inch bolts placed about 2 feet apart alternately near the top and bottom, about a distance of $\frac{d}{4}$ above or below the NA.

In choosing a rolled steel beam for any purpose the architect is constrained to select from the manufacturers' catalogues or from the British Standard lists the section that most closely approximates to the requirements. Such lists usually contain all the properties of the sections tabulated.

In making a selection it is necessary to satisfy three conditions, *i.e.*—

1. The intensity of the stress must not exceed the safe value for the material in tension and compression.

2. The intensity of stress in the web must not be such as to cause buckling.

3. The deflection must be limited to the maximum decided upon for any given purpose.

For the first it is usual to equate the moment of the load with the moment of resistance to determine the modulus of the section.

For the second condition it is necessary to limit the load to the maximum resistance of the web to buckling due to the shearing stress. Failure in this way is possible in short beams with heavy loads, in which case the limit of resistance to buckling will be reached before the value of the modulus of the section.

Thirdly, a suitable figure for the maximum allowable deflection is $\frac{1}{400}$ of the length.

EXAMPLE.—Select a rolled steel beam supported at the ends to carry safely a distributed load of 58 tons over a 10-foot span. Maximum stress to be 8 tons per square inch.

$$Z = \frac{M}{f} \qquad \text{(see p. 554)} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (1)$$

$$= \frac{Wl}{8f} = \frac{58 \times 10 \times 12}{8 \times 8} = 109 \text{ in.}^3$$

Shearing.—A 20″ × 6½″ × 65 lbs. R.S.J. has a section modulus (Z) of 122·6 in.³ As the loading is fairly heavy for the length of span, the question of failure by shear must be considered and it will be found that this beam is not strong enough. Try a 22″ × 7″ × 75 lbs. R.S.J.

Determine the maximum shear stress at the supports. Reaction = 29 tons.

$$\text{Then } q = \frac{F}{Ib_w}\left\{\int_{10\cdot17}^{11} y \,.\, b_f \,.\, dy + \int_{y}^{10\cdot17} y \,.\, b_w \,.\, dy\right\}$$

$$= \frac{29}{1677 \times 0\cdot5}\left\{\frac{7}{2}(11^2 - 10\cdot17^2) + \frac{0\cdot5}{2}(10\cdot17^2 - y^2)\right\}$$

$$= 3\cdot02 - 0\cdot0097\,y^2$$

The stress at the NA where $y = 0$ will be 3·02 tons/in.²

q at 3″	above NA	$= 3\cdot02 - 0\cdot009 \times 3^2 = 2\cdot94$ tons/in.²
q at 6″	,, ,,	$= 3\cdot02 - 0\cdot009 \times 6^2 = 2\cdot70$,,
q at 9″	,, ,,	$= 3\cdot02 - 0\cdot009 \times 9^2 = 2\cdot29$,,
q at 10·17″	,, ,,	$= 3\cdot02 - 0\cdot009 \times 10\cdot17^2 = 2\cdot09$,,

At an infinitely small distance above 10·17 inches the section changes from 0·50 inches to 7 inches, fourteen times greater,

and $$q = \frac{2\cdot09}{14} = 0\cdot15 \text{ tons/in.}^2$$

q at 11″ above NA = 0

The safe allowable shearing stress in the web is obtained by considering the compressional stress caused along a line at 45 degrees to the NA in a strip of unit width multiplied by the thickness of the web and determining the safe value of p as for a strut. Thus the intensity of compression along a line of 45 degrees is 3·02 tons/in.² (see p. 555).

The length of the strip = $20\cdot33\sqrt{2} = 28\cdot8$ inches.

$$\text{Radius of gyration} = \sqrt{\frac{I}{A}} = \sqrt{\frac{\frac{bd^3}{12}}{bd}} = \sqrt{\frac{d^2}{12}} = \sqrt{\frac{0\cdot50^2}{12}}$$

$$= 0\cdot144 \text{ inches.}$$

$$\frac{l}{r} = \frac{28\cdot8}{0\cdot144} = 200.$$

From Rankine's curve the value of p for $\frac{l}{r} = 200$ is 3 tons/in.^{2}

Therefore as the maximum shear is 3·02 tons/in.^{2}, the web is safe.

It is a common approximation with steel girders to consider the shearing stress as distributed uniformly through the gross area of the web, *i.e.*—

$$\text{Mean shearing stress} = \frac{\text{Reaction}}{\text{Web thickness} \times \text{web depth}}$$

$$= \frac{29}{0{\cdot}50 \times 22}$$

$$= 2{\cdot}63 \text{ tons/in.}^2$$

Deflection.—

$$y = \frac{5}{384} \cdot \frac{Wl^3}{EI} \quad (E = 13000 \text{ tons/in.}^2)$$

$$y = \frac{5 \times 58 \times 120 \times 120 \times 120}{384 \times 13000 \times 1677}$$

$$= 0{\cdot}06 \text{ inches.}$$

$$\text{Allowable deflection limit} = \frac{l}{400} = \frac{10 \times 12}{400} = 0{\cdot}3 \text{ inches.}$$

The above is therefore safe.

Example of Plate Girder.—Design a plate girder to carry a distributed load, including the weight of the girder, of 110 tons. Assume effective span equals 50 feet, the depth 4 feet over the backs of the flange angles, and the breadth 1 ft. 6 in. Stress in flanges not to exceed 8 tons per square inch. Stress in web not to exceed 4 tons per square inch.

The flanges include the plates and connecting angles. It is usual to assume that the flanges take the whole of the direct bending stresses and the web the whole of the shearing stresses. One-eighth of the area of the web plate *can* be included in the area of each of the flanges provided that the web plates are efficiently covered at the joints to transmit the horizontal stresses. The depth of the web is taken over the angles.

$$\text{Bending moment} = \frac{Wl}{8} = \frac{110 \times 50 \times 12}{8}$$
$$= 8250 \text{ tons in.}$$
$$\text{Area of flanges A} = \frac{8250}{8 \times 48''}$$
$$= 21{\cdot}5 \text{ sq. inch.}$$

Assume the composition of the flange is as follows :— Use $\frac{7}{8}$-inch diameter rivets for all connections and allow $\frac{1}{16}$ inch extra to the diameter of the rivet for holing. Let the flange angles be $4'' \times 4'' \times \frac{1}{2}''$. Area of two angles less one $\frac{15}{16}$-inch rivet hole each is

$$2\left\{\left(7\tfrac{1}{2} - \frac{15}{16}\right) \cdot \tfrac{1}{2}\right\} = 6{\cdot}56 \text{ sq. in.}$$
$$\therefore \quad \text{Area of plates} = 21{\cdot}5 - 6{\cdot}56$$
$$= 14{\cdot}94 \text{ sq. in.}$$

Effective width of plates less four rivet holes $= 18 - 4 \times \frac{15}{16} = 14{\cdot}25$ inches.

$$\text{Thickness of plates} = \frac{14{\cdot}94}{14{\cdot}25} = 1{\cdot}05 \text{ in.}$$

Let flange be composed of one $\frac{1}{2}$-inch plate along the whole span and one $\frac{5}{8}$-inch plate.

	tons in.
Then M_R of flanges = Angles $6{\cdot}56 \times 48 \times 8$ =	2515
Plates $14{\cdot}94 \times 48 \times 8$ =	5740
	8255

This is slightly in excess of the moment of the load.

The flange plates are curtailed according to the intensity of the stress ; a plate girder thus becomes a close approximation to a girder of uniform strength. The first plate, for practical reasons, is made the full length of the girder. The curtailed length of the second and succeeding plates is found usually by the methods shown on Fig. 434. The curve of the bending moment is set up, in this case a parabola, the altitude being made a scaled length to represent the area of the flange. The areas of the components of the flange are then set up, lines being drawn through parallel

to the base of the figure. The angles and first plate are made the full length. Each succeeding plate is cut off where it intersects the curve of the bending moment. The length of the plates can then be scaled from the diagram. In practice it is usual to extend the plate beyond the theoretical length thus found by about four rows of rivets, or the extension of the plate is sometimes made sufficient to contain enough rivets in single shear to equal the strength of the plate. $\frac{7}{8}$-inch rivet in single shear at 6 tons/inches^2 takes 3·61 tons.

$$\text{Thus strength of plate} = 14{\cdot}25 \times \tfrac{5}{8} \times 8$$
$$= 71{\cdot}3 \text{ tons.}$$
$$\text{Number of rivets} = \frac{71{\cdot}3}{3{\cdot}61}, \text{ say 20 rivets.}$$

Or the extension of the plate is made to take 5 rows of rivets or 1 ft. 8 in. longer than the length obtained from the curve of the bending moment.

Joints in Plates.—The jointing in the flange plates is usually made with single covers (Fig. 431) ; if it is necessary to join more than one plate in the flange, the joints are usually grouped under one cover. The thickness of the cover t.

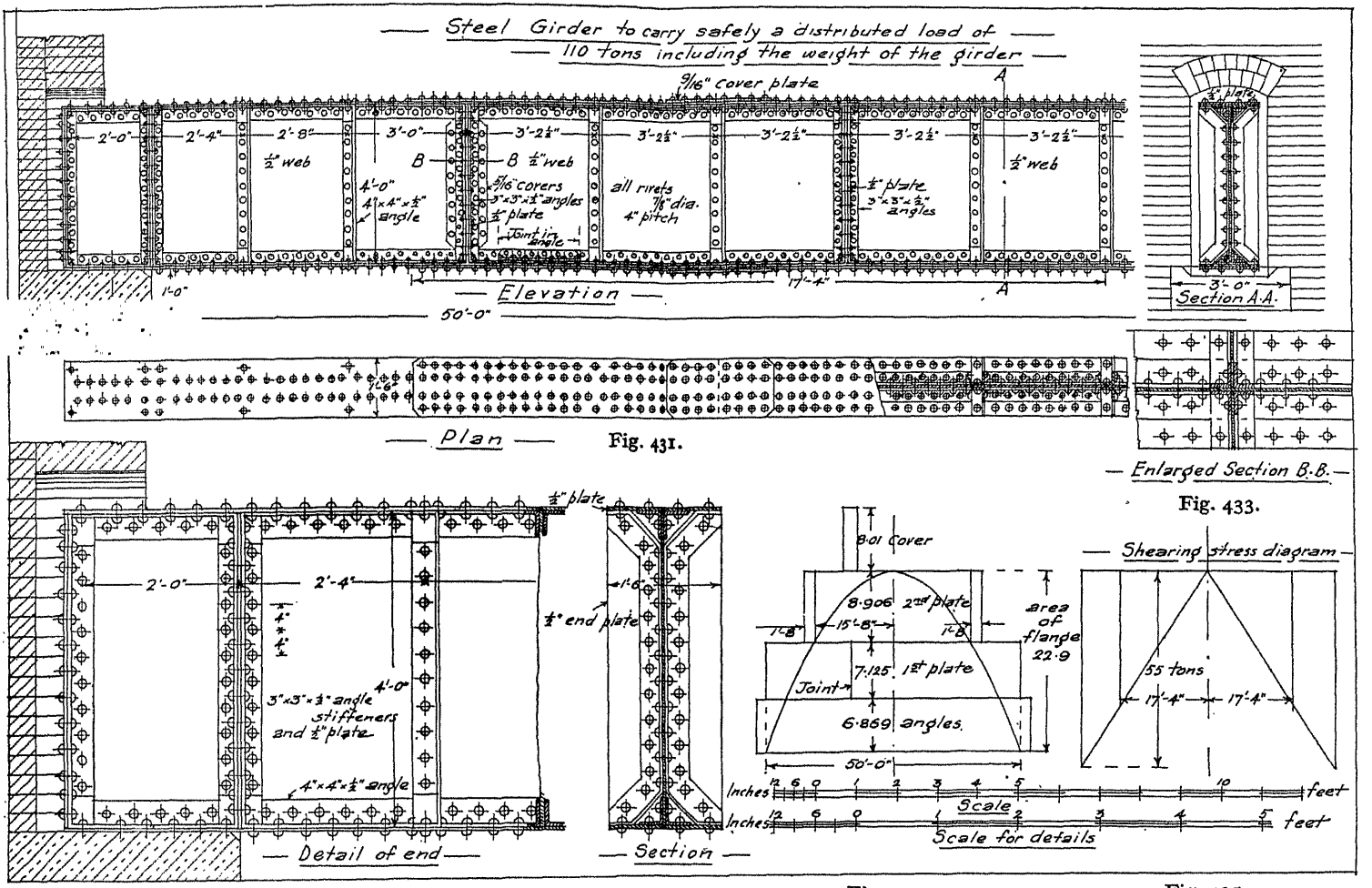
$$t = t_1 + \frac{t_1}{8} \text{ (where } t_1 \text{ is thickness of plate).}$$
$$= \frac{1}{2} + \frac{1}{16}$$
$$= \tfrac{9}{16}'' \text{ cover plate.}$$

In this case let there be a joint in the main plate 17 feet from end of the girder.

$$\text{Then strength of plate} = 14{\cdot}25 \times \tfrac{1}{2} \times 8$$
$$= 57 \text{ tons.}$$
$$\text{Number of rivets each side of joint} = \frac{57}{3{\cdot}61} = \text{nearly } 16.$$

Joint in Angles.—

$$\text{Strength of angles} = \text{A (less 1 hole)} \times 8 \text{ tons/in.}^2$$
$$= 3{\cdot}28 \times 8$$
$$= 26{\cdot}2 \text{ tons.}$$
$$\text{Number of rivets each side of joint} = \frac{26{\cdot}2}{3{\cdot}61} = \text{say } 8.$$



Figs. 431—435.

[*Between pages* 590 *and* 591.

The cover is made of bent plate equal in area to the angles joined, if $\frac{5}{8}$-inch plate be used for angle covers it will give an area slightly in excess of the actual requirements. The joints in the pairs of angles should be broken (Fig. 431).

Web.—The web is assumed to take the whole of the shearing stress. This will be a maximum at the supports in a beam under a distributed load, and zero at the centre, as shown on the shearing stress diagram, Fig. 435.

The shearing stress is usually taken as being uniformly distributed throughout the depth (see previous reference to flanged girders). The web must be thick enough (1) to provide sufficient bearing area for the rivets connecting the web to the flange, (2) also thick enough to resist buckling from the combined tensional and compressional stresses at an angle of 45 degrees to the flanges resulting from the horizontal and vertical shearing stresses. If the web is given a suitable thickness to allow for a reasonable pitch for the rivets it will require to be supplemented by stiffeners to prevent buckling. It is usual to limit the intensity of shearing in the web to about 4 tons per square inch, or about half the direct stresses in the flanges.

Take for purposes of calculation that the full reaction occurs at the inner edge of the bearing surfaces and at intermediate points in proportion. The shearing F may be measured from the diagram (see Fig. 435).

Stiffeners.—These consist of T or L sections used in pairs, one on each side of the web (Figs. 431 and 432). Their functions are: (1) to distribute the concentrated loads at any part throughout the webs and flanges (they may be considered as short struts, and should have an area sufficient to take as their load the total vertical shearing at the point they are placed); (2) to limit the unsupported length of the web so that the latter may safely resist as a strut the thrust at an angle of 45 degrees.

It is usual to place one pair of stiffeners at the extremities of the girder, and a pair at the inner extremities of the bearings.

Assuming that at the point of support the stiffeners

take the whole load, then if p = the full compressional value 8 tons/inches^2, the area required $= \frac{F}{p} = \frac{55}{8} = 6{\cdot}88$ square inches.

If a pair of $3'' \times 3'' \times \frac{3}{8}''$ angles with a $\frac{1}{2}''$ web plate be employed to form the stiffeners at this point, it would give an area in excess of the requirements, and with the usual transverse spacing for the rivets in the angles, give a 4-inch pitch, and thus not interrupt the longitudinal spacing for the rivets. This form of stiffener is also suitable at all joints in the web.

Let the web be $\frac{1}{2}$ inch thick at the supports, t = thickness of web, q = intensity of shearing. Then $q = \frac{F}{Dt} = \frac{55}{48'' \times \frac{1}{2}} = 2{\cdot}29$ tons per square inch.

Pitch of Rivets in Flange Angles.—Let P = pitch of rivets, $P \times qt = R$ = the resistance that must be offered by each rivet.

$$\therefore \quad P = \frac{R}{qt}.$$

Then substituting $\frac{F}{Dt}$ for q,

$$P = \frac{RD}{F}$$

Taking $\frac{7}{8}''$ rivets, R in double shear $= 7{\cdot}23$ tons.
With $\frac{1}{2}''$ plates, ,, bearing $= 5{\cdot}25$ tons.

$$\therefore \quad P = \frac{5{\cdot}25 \times 48}{55} = 4{\cdot}6 \text{ in.}$$

say $4''$ pitch.

The thickness of the web may be reduced or the pitch of the rivets increased towards the centre of the girder where the shearing is less.

The intensity of stress in compression in the web in a direction at 45 degrees to the flanges will be the same as the shear stress in a vertical or horizontal direction.

Consider a 1-inch strip of the web at 45 degrees and of length l as a strut fixed at the ends under a compression stress p of 2·29 tons per square inch equal to the maximum shearing stress in the web.

If the horizontal distance between the stiffeners $= d$ and $t =$ thickness of the web, then, using Rankine's formula for struts,

$$p = \frac{f_c}{1 + a\left(\frac{l}{r}\right)^2} = \frac{7{\cdot}5}{1 + \frac{1}{30000}\left(\frac{l}{r}\right)^2}$$

Let $l^2 = 2d^2$ and $r^2 = \frac{t^2}{12}$ and $t = \frac{1''}{2}$.

Then at the supports $q = 2{\cdot}29 = p$,

and
$$2{\cdot}29 = \frac{7{\cdot}5}{1 + \frac{1}{30000} \times \frac{2d^2}{\frac{t^2}{12}}}$$

$$\frac{2{\cdot}29}{7{\cdot}5} = \frac{1}{1 + {\cdot}0032d^2}$$

$$d^2 = \frac{2{\cdot}27}{{\cdot}0032} = 710$$

$$d = 26{\cdot}6''$$

or, if stiffeners be out of $4'' \times 4'' \times \frac{1}{2}''$ angles, then distance from centre to centre $=$ 2 ft. 4 in.

The spacing of the stiffeners would be variable when computed according to this theory, that is, they would be placed farther apart as the shearing stress became less towards the centre of the beam. In large beams they are placed a distance not exceeding the depth of the beam apart, and in any case, not more than 5 feet.

Then at 2 ft. 4 in. from the supports

$$F = 49{\cdot}9 \text{ tons}$$

$$q = \frac{F}{Dt} = \frac{49{\cdot}9}{48 \times \frac{1}{2}} = 2{\cdot}08 \text{ tons per sq. inch.}$$

$$\therefore \quad 2{\cdot}08 = \frac{7{\cdot}5}{1 + {\cdot}0032d^2}$$

$$\therefore \quad d = 28{\cdot}5''$$

Centre to centre, say 2′ 8″.

Then
$$2' 4'' + 2' 8'' = 5' 0''$$

$$F \text{ at } 5' 0'' = 44 \text{ tons}$$

$$q = \frac{F}{Dt} = \frac{44}{48 \times \frac{1}{2}} = 1{\cdot}83 \text{ tons}$$

$$\therefore \quad 1{\cdot}83 = \frac{7{\cdot}5}{1 + {\cdot}0032d^2}$$

$$\therefore \quad d = 31{\cdot}6''$$

Centre to centre, say 3′ 0″.

Then 2′ 4″ + 2′ 8″ + 3′ 0″ = 8′ 0″

F at 8′ 0″ from the supports = 37·4 tons.

This would be a suitable distance for a joint in the web plate.

The number of rivets N each side of joint $= \frac{F}{R} = \frac{37{\cdot}4}{5{\cdot}25} = 8.$

Use No. 2 $\frac{5}{16}$ inch cover plates with No. 2 angles 3″ × 3″ × ½″ with ½-inch cross web plate as shown on Fig. 431. Using a 4-inch pitch it will give nine rivets through the stiffeners, and eight through the covers, which is in excess of the requirements.

At 8 feet from the supports

$$F = 37{\cdot}4 \text{ tons.}$$

$$q = \frac{F}{Dt} = \frac{37{\cdot}4}{48 \times \frac{1}{2}} = 1{\cdot}56 \text{ tons/in.}^2$$

∴ As before

$$1{\cdot}56 = \frac{7{\cdot}5}{1 + {\cdot}0032d^2}$$

$$d = 34{\cdot}4 \text{ inches.}$$

Centre to centre, say 3′ 2½″.

Let a stiffener occur at mid span and space out the remaining stiffeners equally at 3 ft. 2¼-in. centres.

Make a joint in the web at 2 × 3′ 2½″ = 6 feet 5 inches from the centre.

Shear F at this point is 14·2 tons. Number of rivets required $= \frac{14{\cdot}2}{5{\cdot}25} = 3.$ Use a similar arrangement of angles as at the previous joint but without the cover plates. The number of rivets is in excess of the requirements.

Continuous Beams.—It happens in ferro-concrete work that beams are frequently continuous over several supports. To obtain the stresses and reactions in a beam of this sort it is necessary to employ a different method to that which

has so far been used. The beam must be considered as a whole. It is not possible to consider separately each portion of the beam between the various supports.

Theorem of Three Moments. Consider a section of a continuous beam (Fig. 436) which includes three supports. Let the three bending moments at the supports be M_1, M_2, M_3. Let the loading on the beam be w per unit length.

Write down general expression for the bending moment in each of the two spans.

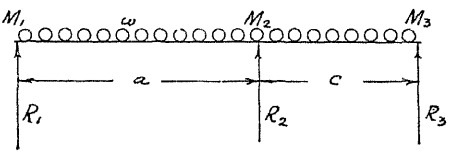


Fig. 436.

$$\text{BM} = \text{EI}\frac{d^2y}{dx^2}$$

$$\text{EI}\frac{d^2y}{dx^2} = M_1 + R_1x - \frac{wx^2}{2} \quad + R_2(x-a) \quad . \quad (1)$$

$$\text{EI}\frac{dy}{dx} = M_1x + R_1\frac{x^2}{2} - \frac{wx^3}{6} + C \quad + R_2.\frac{(x-a)^2}{2} \quad . \quad (2)$$

$$\text{EI}y = M_1\frac{x^2}{2} + R_1\frac{x^3}{6} - \frac{wx^4}{24} + Cx + D \quad + R_2.\frac{(x-a)^3}{6} \quad . \quad (3)$$

Unknown are C, D, R_1, R_2, M_1.

When $x = 0$, $y = 0$ $\quad \therefore \quad D = 0$

" $x = a$, $y = 0$

" $x = a_2$, $y = 0$. $\quad a_2 = a + c$

Substitute in (3) $x = a$

$$0 = M_1\frac{a^2}{2} + R_1\frac{a^3}{6} - \frac{wa^4}{24} + Ca \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (4)$$

Substitute in (3) $x = a_2$

$$0 = M_1\frac{a_2^2}{2} + R_1\frac{a_2^3}{6} - \frac{wa_2^4}{24} + Ca_2 + R_2.\frac{(a_2-a)^3}{6} \quad . \quad (5)$$

Substitute in (1) $x = a$, where $\text{BM} = M_2$

$$\therefore \quad M_2 = M_1 + R_1a \quad \frac{wa^2}{2} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (6)$$

Substitute in (1) $x = a_2$, where BM $= M_3$

$$\therefore \quad M_3 = M_1 + R_1 a_2 - \frac{w a_2^2}{2} + R_2(a_2 - a) \quad . \quad . \quad . \quad (7)$$

Multiply equation (4) by a_2 and equation (5) by unity and subtract one from the other. Equation (4) can be divided through by a.

$$(4) \div a \qquad 0 = M_1 \frac{a}{2} + R_1 \frac{a^2}{6} - \frac{wa^3}{24} + C$$

$$(4) \times a_2 \qquad 0 = M_1 . \frac{a a_2}{2} + R_1 \frac{a^2 a_2}{6} - \frac{wa^3 a_2}{24} + C a_2$$

$$(5) \times 1 \qquad 0 = M_1 \frac{a_2^2}{2} + R_1 \frac{a_2^3}{6} - \frac{w a_2^4}{24} + C a_2 + R_2 \frac{(a_2 - a^3)}{6}$$

Subtracting,

$$0 = \frac{M_1 a_2}{2}(a_2 - a) + \frac{R_1 a_2}{6}(a_2^2 - a^2) - \frac{w a_2}{24}(a_2^3 - a^3) + R_2 . \frac{(a_2 - a)^3}{6} \quad . \quad . \quad . \quad . \quad (8)$$

Eliminate between (6), (7), (8).

From (8)

$$M_1 = \frac{R_1}{3} . (a_2 + a) - \frac{w}{12}(a_2^2 + a a_2 + a^2) + R_2 . \frac{(a_2 - a)^2}{3 a_2} \quad . \quad (9)$$

From (6)

$$- M_1 + M_2 = R_1 a - \frac{w a^2}{2} \quad . \quad . \quad . \quad . \quad . \quad (10)$$

From (7)

$$- M_1 + M_3 = R_1 a_2 - \frac{w a_2^2}{2} + R_2(a_2 - a) \quad . \quad . \quad (11)$$

Eliminate R_1 and R_2 between above three equations.

Multiply (9) by $-3a_2$
" (10) " $2a_2$
" (11) " $(a_2 - a_1)$

Thus—

$$3a_2 M_1 = -R_1 a_2(a_2 + a) + \frac{w a_2}{4}(a_2^2 + a a_2 + a^2) - R_2(a_2 - a)^2 \quad . \quad . \quad (12)$$

$$- 2a_2 M_1 + 2a_2 M_2 = 2a_2 a R_1 - w a_2 a^2 \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (13)$$

$$- M_1(a_2 - a) + M_3(a_2 - a) = R_1 a_2(a_2 - a) - \frac{w a_2^2}{2}(a_2 - a) + R_2(a_2 - a)^2 \quad . \quad . \quad . \quad . \quad (14)$$

Add equations (12), (13), (14) together.

$$aM_1 + 2a_2M_2 + (a_2 - a)M_3$$

$$= -\frac{w}{4}(-a_2^3 - aa_2^2 - a_2a^2 + 4a_2a^2 + 2a_2^3 - 2a_2^2a)$$

$$= -\frac{w}{4}(a_2^3 - 3a_2^2a + 3a_2a^2)$$

$$= -\frac{w}{4}(a_2^3 - 3a_2^2a + 3a_2a^2 - a^3 + a^3)$$

$$aM_1 + 2a_2M_2 + (a_2 - a)M_3 = -\frac{w}{4}\left\{(a_2 - a)^3 + a^3\right\}$$

This equation is usually written in this form:

$$aM_1 + 2(a + c)M_2 + cM_3 = -\frac{w}{4}(a^3 + c^3)$$

This formula connects the three moments.

If there are a number of supports, say n, one can obtain $n - 2$ sets of equations similar to the above by combining the bending moments at the supports in groups of three as in the equation. There are n supports, and hence n values of M. Two more conditions are necessary in order that all these values may be determined. These are given by the end conditions. Thus if the ends of the beam are free the bending moment at the ends is zero.

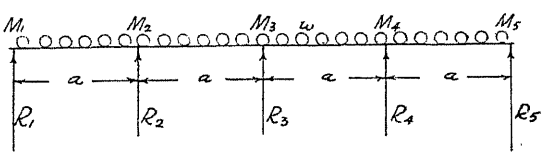


Fig. 437.

Consider the case of a beam continuous over four spans (Fig. 437). It is required to evaluate the bending moments over the supports and the five reactions. The beam is freely supported at the ends.

Consider the first three supports starting from the left-hand side and substitute the particular values given in the equation above.

Thus

$$aM_1 + 4M_2a + aM_2 = -\frac{2wa^3}{4}$$

$$\text{or } M_1 + 4M_2 + M_3 = -\frac{wa^2}{2} \quad \ldots\ldots (1)$$

Consider also the three moments $M_2M_3M_4$ and $M_3M_4M_5$. Connect in a similar manner to above.

$$M_2 + 4M_3 + M_4 = -\frac{wa^2}{2} \quad \ldots\ldots (2)$$

$$M_3 + 4M_4 + M_5 = -\frac{wa^2}{2} \quad \ldots\ldots (3)$$

Take equations (1) and (2). Since the ends of the beam are freely supported M_1 and M_5 are zero.

Also notice that from symmetry

$$M_2 = M_4$$
$$R_2 = R_4$$
$$R_1 = R_5$$

Equation (1) becomes $4M_2 + M_3 = -\frac{wa^2}{2} \quad \ldots\ldots (4)$

Equation (2) becomes $2M_2 + 4M_3 = -\frac{wa^2}{2} \quad \ldots\ldots (5)$

Solving simultaneously

$$4M_2 + M_3 = -\frac{wa^2}{2}$$

$$4M_2 + 8M_3 = -wa^2$$

$$\therefore\ 7M_3 = +\frac{wa^2}{2} - wa^2$$

$$\therefore\ M_3 = -\frac{wa^2}{14}$$

Substitute this value in (4)

$$4M_2 + M_3 = -\frac{wa^2}{2}$$

$$4M_2 - \frac{wa^2}{14} = -\frac{wa^2}{2}$$

$$\therefore\ M_2 = -\frac{3wa^2}{28} = M_4.$$

The values for the reactions must now be found. Write

down the general expressions for the bending moment at any point in the first three sections as in previous examples.

$$BM = R_1 x - \frac{wx^2}{2} \Big| + R_2 (x - a) \Big| + R_3 (x - 2a)$$

The first part of the equation represents the bending moment in the first span at any point. M_2 equals the bending moment at the end of the first span. Hence

$$M_2 = R_1 a - \frac{wa^2}{2}$$

$$-\frac{3wa^2}{28} = R_1 a - \frac{wa^2}{2}$$

$$\therefore \quad R_1 a = \frac{11wa^2}{28}$$

$$\therefore \quad R_1 = R_5 = \frac{11wa}{28}$$

Similarly as M_3 is at the end of the second span.

$$M_3 = 2R_1 a - 2wa^2 + R_2 a = -\frac{wa^2}{14}$$

$$\frac{11wa^2}{14} - 2wa^2 + R_2 a = -\frac{wa^2}{14}$$

$$\therefore \quad R_2 = \frac{16wa}{14}$$

$$\therefore \quad R_2 = R_4 = \frac{8wa}{7}.$$

Again, to determine R_3.

$$M_4 = 3R_1 a - \frac{9wa^2}{2} + 2R_2 a + R_3 a = -\frac{3wa^2}{28}$$

$$= \frac{33wa^2}{28} - \frac{9wa^2}{2} + \frac{16wa^2}{7} + R_3 a = -\frac{3wa^2}{28}$$

$$\therefore \quad R_3 = \frac{13wa}{14}.$$

The values for the reactions and the bending moments at the supports for cases having up to five clear spans have been worked out and are tabulated below. See Figs. 438 to 441.

Two spans.

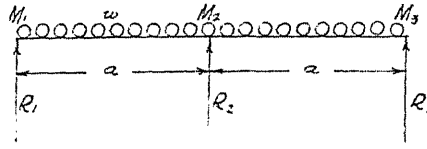


Fig. 438.

$$M_1 = M_3 = 0 \qquad R_1 = R_3 = \frac{3wa}{8}$$

$$M_2 = \frac{wa^2}{8} \qquad R_2 = \frac{10wa}{8}$$

Three spans.

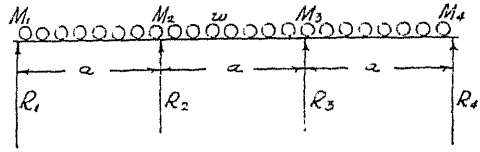


Fig. 439.

$$M_1 = M_4 = 0 \qquad R_1 = R_4 = \frac{2wa}{5}$$

$$M_2 = M_3 = \frac{wa^2}{10} \qquad R_2 = R_3 = \frac{11wa}{10}$$

Four spans.

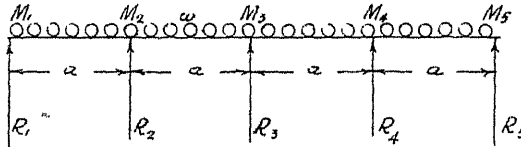


Fig. 440.

$$M_1 = M_5 = 0 \qquad R_1 = R_5 = \frac{11wa}{28}$$

$$M_2 = M_4 = \frac{3wa^2}{28} \qquad R_2 = R_4 = \frac{8wa}{7}$$

$$M_3 = \frac{wa^2}{14} \qquad R_3 = \frac{13wa}{14}$$

Five spans.

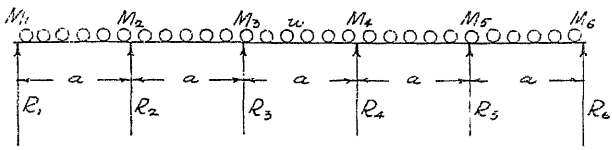


Fig. 441.

$$M_1 = M_6 = 0 \qquad R_1 = R_6 = \frac{15wa}{38}$$

$$M_2 = M_5 = \frac{2wa^2}{19} \qquad R_2 = R_5 = \frac{43wa}{38}$$

$$M_3 = M_4 = \frac{3wa^2}{38} \qquad R_3 = R_4 = \frac{37wa}{38}$$

EXAMPLE.—Consider the case of a staircase beam which is to be built in ferro-concrete (Figs. 442 to 444). For calculation purposes the beam can be considered as straight and not cranked. $w = 20$ lbs./inch run. One foot width considered.

Employ theorem of three moments.

$$aM_1 + 2(a + c)M_2 + cM_3 = -\frac{w}{4}(a^3 + c^3).$$

Then

$$60M_1 + 2(60 + 100)M_2 + 100M_3 = -\frac{20}{4}(60^3 + 100^3).$$

Then as $M_1 = 0$

$$320M_2 + 100M_3 = -6080000 \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (1)$$

Also considering the three moments $M_2M_3M_4$.

$$100\,M_2 + 2(100 + 60)M_3 + 60\,M_4 = -\frac{20}{4}(100^3 + 60^3)$$

$$M_4 = 0.$$

$$100\,M_2 + 320\,M_3 = -6080000 \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (2)$$

Solve for M_3 between (1) and (2).
Multiply equation (1) by unity.
,, ,, (2) ,, 3·2.

Thus

$$320\,M_2 + 100\,M_3 = -6080000$$
$$320\,M_2 + 1024\,M_3 = -19456000$$

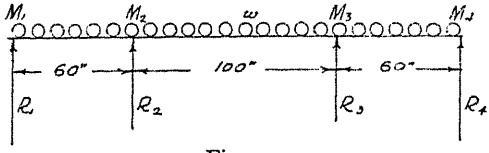


Fig. 442.

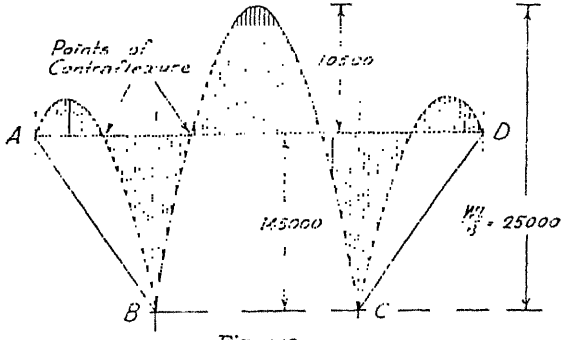


Fig. 443.

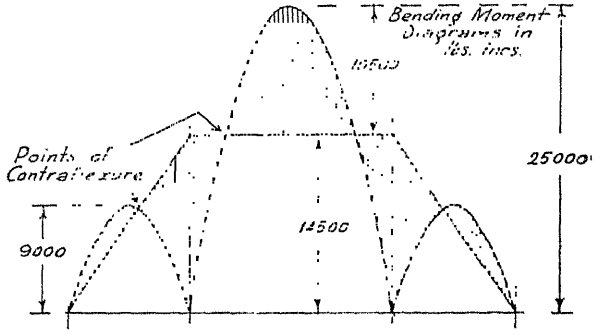


Fig. 444.

Figs. 442—444.

Subtract

$$924\,M_3 = -13376000$$

$$M_3 = -14476 \text{ lbs. in.}$$

$$\therefore \quad M_2 = -14476 \text{ lbs. in.}$$

Approx. $M_2 = M_3 = -14500$ lbs. in.

Next determine the reactions.

Write general expression for bending moment in each of the three sections of beam.

$$BM = R_1 x - \frac{wx^2}{2} \Big| + R_2(x - 60) \Big| + R_3(x - 160) \quad . \quad . \quad (3)$$

When $x = 60$, $BM = M_2 = -14476$ lbs. in.
$\therefore$ Substituting $x = 60$ in.

$$-14476 = R_1 . 60 - \frac{20 \times 60^2}{2}$$

$$60\, R_1 = -14476 + 36000 = 21524$$

$$R_1 = 359 \text{ lbs.}$$

$$\therefore \quad R_4 = 359 \text{ lbs.}$$

$$\therefore \quad R_3 = R_2 = 1841 \text{ lbs.}$$

It is now possible to calculate the bending moment at any part of the above beam. Determine its value at the middle of the central span. Substitute $x = 110$ in the appropriate section of equation (3).

$$BM = R_1 x - \frac{wx^2}{2} + R_2(x - 60)$$

$$= 359 \times 110 - \frac{20 \times 110^2}{2} + 1841\,(110 - 60)$$

$$= 10500 \text{ lbs. in.}$$

In addition and as a check on the above method it is possible, after the moments at the supports have been found, to draw the bending moment diagram and obtain other moments from the diagram. In many cases where only the maximum moments are required, this method will be found easier and quicker.

The completed diagram is shown in Fig. 443. Set down the value of M_2 and M_3 and draw the three straight lines AB, BC, CD. Now consider the three sections of the beam as entirely separate and set up on AB, BC, CD the three parabolas which would represent the bending moment diagrams for beams simply supported and carrying uniformly distributed loads, and in which case bending moment at centre equals $\frac{Wl}{8}$. The diagram now shows the bending moment along the beam, the hatched portions representing the bending moment.

This diagram has been drawn in a manner similar to previous examples having AD, the base line, horizontal.

It is common practice in this type of case to arrange the diagram somewhat differently as shown in the alternative bending moment diagram in Fig. 444.

The calculations required are as follows:

Centre span

$$\frac{Wl}{8} = \frac{wl^2}{8} = \frac{20 \times 100^2}{8} = 25000 \text{ lbs. in.}$$

BM at centre $= 25000 - M_2$
$= 25000 - 14500$
$= 10500$ lbs. in.

This figure corresponds to the figure obtained mathematically.

The points of contraflexure and any other points required are most easily obtained from an accurate diagram, but can be obtained if required by the use of equation (3) above.

EXAMPLE.—Consider a beam such as would occur in the ferro-concrete floor illustrated in Fig. 445.

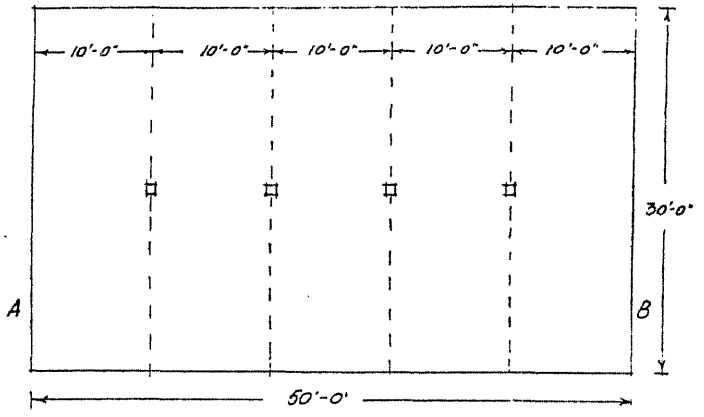


Fig. 445.

In designing the slab which covers the whole area a strip 1 foot wide would be considered as running from wall A to wall B. This practically constitutes a beam continuous over five spans, and the reactions on the various

beams would have the values as shown in Fig. 441. It will be seen that the first beam carries the greatest load and this equals $\frac{43wa}{38}$.

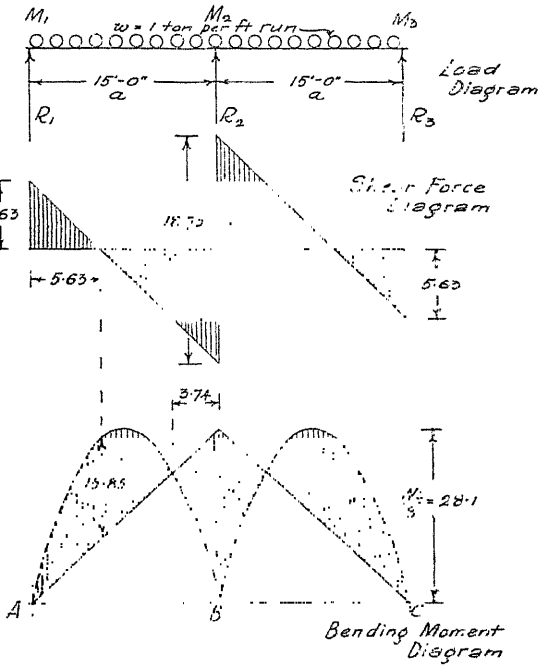


Fig. 448.

Figs. 446—448.

Assume that the superimposed load on the floor is 100 lbs./square foot and that the dead load due to the weight of the concrete is also 100 lbs./square foot.

$\therefore$ Total load on 1′ 0″ strip $= 50 \times 1 \times 200 = 10000$ lbs.

$$\therefore \quad w = \frac{10000}{50} = 200 \text{ lbs./ft. run.}$$

Reaction on first beam (Figs. 446 to 448). $R = \frac{43wa}{38}$

$$\therefore \quad R = \frac{43 \times 200 \times 10}{38} = 2260 \text{ lbs.}$$

say 1 ton.

$\therefore$ Load on beam = 1 ton per foot run

From Fig. 446 we have

$$M_1 = M_3 = 0 \qquad R_1 = R_3 = \frac{3wa}{8}$$

$$M_2 = \frac{wa^2}{8} \qquad R_2 = \frac{10wa}{8}$$

$$\therefore \quad M_2 = \frac{1 \times 15^2}{8} = 28{\cdot}1 \text{ tons ft.}$$

$$R_1 = R_3 = \frac{3 \times 1 \times 15}{8} = 5{\cdot}63 \text{ tons.}$$

$$R_2 = \frac{10 \times 1 \times 15}{8} = 18{\cdot}75 \text{ tons.}$$

Now draw bending moment diagram as in Fig. 448. Set up $M_2 = 28{\cdot}1$ tons feet. On AB, BC set up parabolæ representing the BM curve if the beams were simply supported at the ends, the central ordinates of which equal $\frac{Wl}{8}$.

$$\frac{Wl}{8} = \frac{1 \times 15 \times 15}{8} = 28{\cdot}1 \text{ tons ft.}$$

The BM diagram is shown hatched and the bending moment at any point on the beam can be found from the diagram.

The shear force diagram is also shown in Fig. 447.

The maximum bending moment between the supports occurs where the shear force is zero.

Write equation for sheer force in left-hand bay.

$$F = R_1 - wx.$$

Equate to zero

$$R_1 - wx = 0$$

$$wx = R_1$$

$$x = \frac{R_1}{w} = \frac{5{\cdot}63}{1} = 5{\cdot}63 \text{ ft.}$$

$\therefore$ Maximum bending moment occurs 5·63 feet from end support.

Write equation for bending moment in left-hand bay.

$$B = R_1x - \frac{wx^2}{2}. \qquad \text{Substitute } x = 5{\cdot}63.$$

$$= 5{\cdot}63 \times 5{\cdot}63 - \frac{1 \times 5{\cdot}63^2}{2}$$

$$= 15{\cdot}85 \text{ tons ft.}$$

$\therefore$ Max. BM between supports = 15·85 tons feet.
The point of contraflexure occurs where BM = 0.

$$B = R_1 x - \frac{wx^2}{2} = 0$$

$$\frac{wx^2}{2} = R_1 x$$

$$\therefore \quad \frac{x}{2} = 5{\cdot}63$$

x = 11·26 feet from end support or 3·74 feet from centre support.

$\therefore$ Point of contraflexure occurs 3·74 feet from centre support.

Example as above. Part 2.—Consider the same beam but assume that only the left-hand bay is loaded. Figs. 449 to 451. The dead load will be over both spans as before.

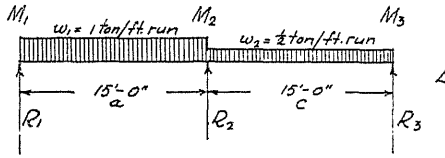


Fig. 449.

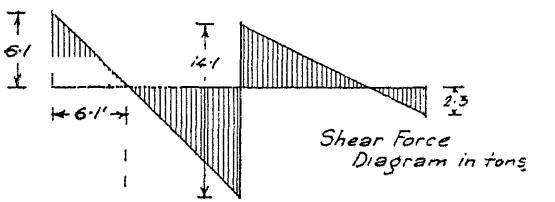


Fig. 450.

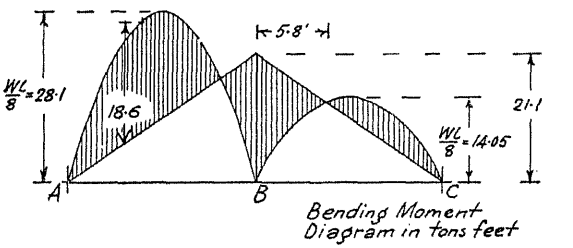


Fig. 451.

Figs. 449—451.

The dead load and superimposed load will be treated separately.

Use equation of three moments to determine M_2.

Let w_1 = weight per foot run on left-hand span.
" w_2 = " " " right- " "

$$aM_1 + 2(a + c)M_2 + cM_3 = -\tfrac{1}{4}\{w_1a^3 + w_2c^3\}$$

Now $M_1 = M_3 = 0$ and $w_1 = 1$ ton, $w_2 = \frac{1}{2}$ ton.

$\therefore$ Substituting

$$2(15 + 15)M_2 = -\tfrac{1}{4}\{(1 + 15^3) + (\tfrac{1}{2} \times 15^3)\}$$

$$\therefore \quad M_2 = -21{\cdot}1 \text{ tons ft.}$$

To determine reactions,

Write down general expression for bending moment.

$$B = R_1x - \frac{w_1x^2}{2} \text{ in first span.}$$

When $x = 15$. $B = M_2 = -21{\cdot}1$

$$\therefore \quad R_1x - \frac{w_1x^2}{2} = -21{\cdot}1$$

$$15R_1 = -21{\cdot}1 + \frac{15^2}{2}$$

$$R_1 = 6{\cdot}1 \text{ tons.}$$

Measuring x from R_3

$$B = R_3x - \frac{w_2x^2}{2}.$$

When $x = 15$, $B = M = -21{\cdot}1$

$$R_3x = \frac{w_2x^2}{2} = -21{\cdot}1$$

$$15R_3 = -21{\cdot}1 + \frac{15^2}{4}$$

$$R_3 = 2{\cdot}3 \text{ tons.}$$

$\therefore$ by subtraction $R_2 = 14{\cdot}1$ tons.

Thus $R_1 = 6{\cdot}1$ tons.
$R_2 = 14{\cdot}1$ tons.
$R_3 = 2{\cdot}3$ tons.

Now draw BM diagram as in Fig. 451. Set up $M_2 = -21{\cdot}1$ and on the lines AB, BC set up parabolæ, the central ordinates of which equal $\frac{Wl}{8}$ for their respective spans.

First span ... $\frac{Wl}{8} = \frac{1 \times 15 \times 15}{8} = 28{\cdot}1$ tons ft.

Second span ... $\frac{Wl}{8} = \frac{1 \times 15 \times 15}{2 \times 8} = 14{\cdot}05$ tons ft.

The bending moment diagram is shown hatched and from this diagram all necessary information can be measured.

The maximum bending moment occurs where the shear force is zero.

Write equation for sheer force in left-hand bay.

$$F = R_1 - wx.$$

Equate to zero.

$$R_1 - wx = 0.$$
$$6{\cdot}1 - x = 0$$
$$\therefore \quad x = 6{\cdot}1 \text{ ft.}$$

Write equation for bending moment in left-hand bay and make $x = 6{\cdot}1$ feet.

$$B = R_1 x - \frac{wx^2}{2}$$
$$= 6{\cdot}1 \times 6{\cdot}1 - \frac{1 \times 6{\cdot}1^2}{2}$$
$$= 18{\cdot}6 \text{ tons feet.}$$

$\therefore$ Maximum BM between supports = 18·6 tons ft.

It will be noticed that the bending moment in the span is greater when one bay is loaded than when both are loaded.

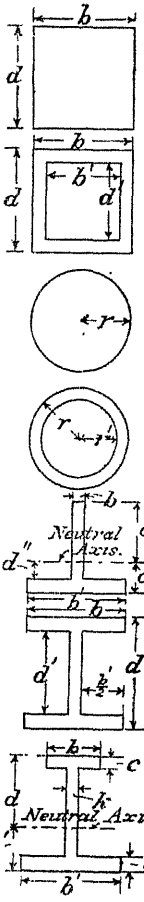
Further, the point of contraflexure in the second bay is nearer the centre of the span in this case and would necessitate carrying the top steel farther along the beam.

For point of contraflexure

$$B = R_3 x - \frac{wx^2}{2} = 0$$
$$= 2{\cdot}3x - \frac{1 \times x^2}{2 \times 2} = 0$$
$$\therefore \quad x = 9{\cdot}2 \text{ ft.}$$
$$\text{or } 15 - 9{\cdot}2 = 5{\cdot}8 \text{ ft. from } R_2.$$

From the above results it would appear that such a beam should be calculated and designed for the worst case, which is when one span is loaded and the other is not.

NUMERICAL VALUE OF THE MOMENTS OF INERTIA OR I., ABOUT A HORIZONTAL NEUTRAL AXIS



$$I = \frac{bd^3}{12}$$

$$I = \frac{bd^3 - b'd'^3}{12}$$

$I = \cdot 7854r^4$ or $\frac{\pi d^4}{64}$ about any transverse axis.

$I = \cdot 7854(r^4 - r'^4)$ or $\frac{\pi}{64}(d^4 - d'^4)$ about any transverse axis.

$$I = \tfrac{1}{3}\{bd^3 + b'd'^3 - (b' - b)d''^3\}$$

$$I = \frac{bd^3 - b'd'^3}{12}$$

$$I = \tfrac{1}{3}\{ba^3 - (b-k)(d-c)^3 + b'd'^3 - (b'-k)(d'-c')^3\}$$

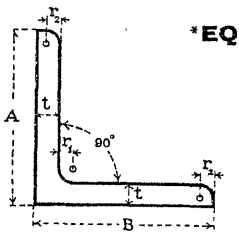
Fig. 452.

YOUNG'S MODULUS, OR THE MODULUS OF ELASTICITY OR VALUE OF E.

	lbs. per square inch.
Cast Iron	17,000,000
Wrought Iron...	29,000,000
Steel, Mild	30,000,000
Oak	1,000,000
Northern Pine	1,000,000 to 1,400,000

List 1

BRITISH STANDARD SECTIONS.



*EQUAL ANGLES.

a = Sectional Area in square inches.

w = Weight in lb. per foot = 3·4 a

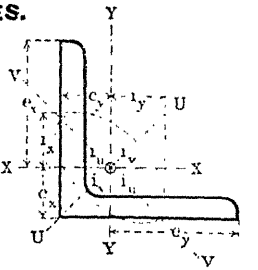
1	2	3	4	5	6 | 7	8	9
Reference No and Code Word	Size. A × B	Standard Thickness. t	Maximum Thickness Rolled.	Minimum Thickness Rolled.	Radii Root. r_1 | Toe. r_2	Calculated Weight per foot. w	Sectional Area a
NBSEA 1 Zabaw — Zabbi	inches. ¾ × ¾	inches ·125 — ·250	inches. ·250	inches. ·125	inches. ·17 ·12	lb. ·58 — 1·06	inches² 172 — ·313
NBSEA 2 Zabco — Zabdu	1 × 1	·125 — ·250	·300	·125	·18 ·13	·80 — 1·49	·234 — ·437
NBSEA 3 Zabex — Zaboz	1¼ × 1¼	·125 — ·250	·300	·125	·20 ·14	1·01 — 1·91	·297 — ·563
NBSEA 4 Zabub — Zabya	1½ × 1½	·125 — ·250	·350	·125	·21 ·15	1·22 — 2·34	·359 — ·687
NBSEA 5 Zabze — Zacax	1¾ × 1¾	·176 — ·300	·375	·175	·23 ·16	1·98 — 3·26	·582 — ·960
NBSEA 6 Zacbe — Zacci	2 × 2	·175 — ·300	·400	·175	·24 ·17	2·27 — 3·77	·669 — 1·110
NBSEA 7 Zacdo — Zacey	2¼ × 2¼	·175 — 300	·450	·175	·26 ·18	2·57 — 4·29	·757 — 1·261
NBSEA 8 Zacfu Zaciz Zacob	2½ × 2½	·250 ·375 ·500	·500	·200	·27 ·19	4·04 5·90 7·65	1·188 1·735 2·250

*See remarks at end of Table.

List 1

BRITISH STANDARD SECTIONS.

EQUAL ANGLES.



c_x c_y Distance of Centre of Gravity from back lines of Angle.

$J = ai^2$ Moment of Inertia.

$i = \sqrt{\frac{J}{a}}$ Radius of Gyration.

e_x e_y Distance of outer fibres from X and Y axes.

$Z = \frac{J}{e}$ Modulus of Section.

10	11	12	13	14	15	16	17	18	19	20	21	22
Centre of Gravity.		Moments of Inertia.				Radii of Gyration.				Moduli of Section.		Reference No.
c_x	c_y	J_x	J_y	Max. J_u	Min. J_v	i_x	i_y	Max i_u	Min. i_v	Z_x	Z_y	
inches.		inches^4		inches^4.		inches.		inches.		inches^3		
·224	·224	·008	·008	·012	·003	·214	·214	·269	·139	·015	·015	NBSEA 1
·271	·271	·013	·013	·020	·007	·208	·208	·255	·145	·028	·028	
·285	·285	·020	·020	032	·008	·292	·292	·368	·189	·028	·028	NBSEA 2
·334	·334	·035	·035	·055	·016	·284	·284	·354	·190	·053	·053	
·346	·346	·041	041	·065	·017	·371	·371	·467	·239	·045	·045	NBSEA 3
·396	·396	·074	·074	·116	·032	·362	·362	·454	·238	·086	·086	
·405	·405	·072	·072	·115	·030	·449	·449	·566	·289	·066	·066	NBSEA 4
·458	·458	·134	·134	211	·056	·441	·441	·554	·286	·128	·128	
·489	·489	·161	·161	·255	·066	·526	·526	·662	·338	·127	·127	NBSEA 5
·539	·539	·255	·255	·402	·108	·515	·515	·647	·335	·210	·210	
·549	·549	·246	·245	·388	·101	604	·604	·762	·388	·169	·169	NBSEA 6
·601	·601	·392	·392	·619	·164	·594	·594	747	384	·280	·280	
·610	·610	·353	·353	·561	·146	·683	·683	·861	·439	·215	·215	NBSEA 7
·662	·662	·571	·571	·904	·237	·673	·673	·847	·434	·359	·359	
·704	·704	·680	·680	1·080	·280	·757	·757	·954	·485	·379	·379	NBSEA 8
·753	·753	·962	·962	1·521	·402	·745	·745	·936	·482	·550	·550	
·799	·799	1·206	1·206	1·891	·521	732	·732	·917	·481	·709	·709	

List 1 (*cont.*)

BRITISH STANDARD SECTIONS.

EQUAL ANGLES.

(*continued*)

1	2	3	4	5	6 | 7	8	9
Reference No. and Code Word	Size. A × B	Standard Thickness. t	Maximum Thickness Rolled.	Minimum Thickness Rolled.	Radii. Root. r_1 | Toe. r_2	Calculated Weight per foot w	Sectional Area. a
	inches.	inches.	inches.	inches.	inches.	lb.	inches²
NBSEA 9		·250				4·46	1·313
Zacuc	2¾ × 2¾	·375	·525	·225	·29 ·20	6·54	1·923
Zacza		·500				8·50	2·501
Zaday							
NBSEA 10		·250				4·89	1·438
Zadba	3 × 3	·375	·525	·250	·30 ·21	7·17	2·110
Zadce		·500				9·35	2·750
Zaddi							
NBSEA 11		·300				6·84	2·011
Zadez	3½ × 3½	·425	·575	·275	·33 ·23	9·50	2·795
Zadfo		·500				11·05	3·251
Zadgu							
NBSEA 12		·300				7·86	2·311
Zadib	4 × 4	·425	·625	·300	·36 ·25	10·95	3·220
Zadoc		·500				12·75	3·751
Zadud							
NBSEA 13		·375				12·28	3·611
Zafaz	5 × 5	—	·700	·350	·42 ·29	—	—
—		·500				16·16	4·752
Zafca							
NBSEA 14		·450				17·67	5·197
Zafde	6 × 6	—	·775	·425	·48 ·34	—	—
—		·625				24·17	7·109
Zafeb							
NBSEA 15		·550				28·90	8·499
Zaffi	8 × 8	—	·950	·550	·60 ·42	—	—
—		·750				38·89	11·439
Zafgo							
NBSEA 16		·550				32·64	9·600
Zafhu	9 × 9	—	1·000	·550	·66 ·46	—	—
—		·750				44·00	12·940
Zafic							

Remarks.

The dimensions, thickness and profile of Standard Equal Angles shall be in accordance with sections in which the angle between the flanges does [illegible] to the Standard.

Angles ordered to the standard thickness shall be practically accurate in profile; but if the thickness is between, above or below the Standards, the flanges will be proportionately longer or shorter than the Standards. [illegible] the radii and the root and toe will remain unchanged. In Equal Sided Angles the thickness of the flanges will be the same.

Angles may be ordered by width of flanges and thickness, or by width of flanges and weight per foot, but not by both thickness and weight per foot. Where thickness is employed in ordering, decimals of an inch shall be used. The Association suggests that all Angles be ordered by size of flanges and weight per foot.

List 1 (*cont.*)

BRITISH STANDARD SECTIONS.

EQUAL ANGLES.

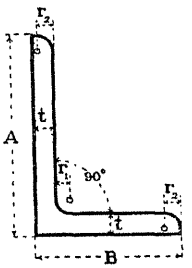
(*continued*)

10	11	12	13	14	15	16	17	18	19	20	21	22
Centre of Gravity.		Moments of Inertia.				Radii of Gyration.				Moduli of Section.		Reference No.
c_x	c_y	J_x	J_y	Max. J_u	Min. J_v	i_x	i_y	Max. i_u	Min. i_v	Z_x	Z_y	
inches.		inches⁴.		inches⁴.		inches.		inches.		inches³.		
·765	·765	·917	·917	1·457	·377	·836	·836	1·053	·536	·462	·462	NBSEA 9
·815	·815	1·304	1·304	2·066	·542	·824	824	1·037	·531	·674	·674	
·862	·862	1·644	1·644	2·587	·702	·811	·811	1·017	·530	·871	·871	
·826	·826	1·203	1·203	1·913	·494	·915	·915	1·153	·586	·553	·553	NBSEA 10
·877	·877	1·720	1·720	2·728	·712	·903	·903	1·137	·581	·810	·810	
·924	·924	2·178	2·178	3·436	·920	·890	·890	1·118	·578	1·049	1·049	
·969	·969	2·296	2·296	3·651	·941	1·069	1·069	1·348	·684	·907	·907	NBSEA 11
1·019	1·019	3·118	3·118	4 947	1·288	1·056	1·056	1·330	·679	1·257	1·257	
1·048	1·048	3·571	3·571	5·653	1·489	1·048	1·048	1·319	·677	1·456	1·456	
1·091	1·091	3·478	3·478	5·532	1·424	1·227	1·227	1·547	·785	1·196	1·196	NBSEA 12
1·142	1·142	4·752	4·752	7·550	1·963	1·215	1·215	1·531	·779	1·663	1·663	
1·172	1·172	5·461	5·461	8·684	2·259	1·207	1·207	1·520	·776	1·931	1·931	
1·367	1·367	8·527	8·527	13·568	3·487	1·537	1·537	1·938	·983	2·347	2·347	NBSEA 13
—		—	—	—	—	—	—	—	—	—	—	
1·418	1·418	11·037	11·037	17·546	4·527	1·524	1·524	1·922	·976	3·081	3·081	
1·642	1·642	17·700	17·700	28·170	7·229	1·845	1·845	2·328	1·179	4·061	4·061	NBSEA 14
—		—	—	—	—	—	—	—	—	—	—	
1·713	1·713	23·734	23·734	37·728	9·740	1·827	1·827	2·304	1·170	5·536	5·536	
2·172	2·172	51·834	51·834	82·522	21·145	2·470	2·470	3·116	1·577	8·894	8·894	NBSEA 15
—		—	—	—	—	—	—	—	—	—	—	
2·254	2·254	68·583	68·583	109·110	28·057	2·448	2·448	3·088	1·566	11·936	11·936	
2·416	2·416	74·525	74·525	118·649	30·400	2·786	2·786	3·516	1·780	11·319	11·319	NBSEA 16
—		—	—	—	—	—	—	—	—	—	—	
2·500	2·500	99·019	99·019	157·618	40·420	2·766	2·766	3·490	1·767	15·233	15·233	

List 2

BRITISH STANDARD SECTIONS.

* UNEQUAL ANGLES.



a = Sectional Area in square inches.

w = Weight in lb. per foot = 3·4 a

1	2	3	4	5	6 | 7		8	9
Reference No and Code Word	Size A × B	Standard Thickness. t	Maximum Thickness Rolled.	Minimum Thickness Rolled.	Radii Root r_1	Radii Toe r_2	Calculated Weight per foot. w	Sectional Area a
	inches.	inches.	inches.	inches.	inches.		lb.	inches²
NBSUA 1		·175					1·98	·582
Zafod	2 × 1½	—	·375	·150	·23	·16	—	—
Zafuf		·300					3·26	·960
NBSUA 2		·175					2·27	·669
Zagab	2½ × 1½	—	·400	·150	·24	·17	—	—
Zagda		·300					3·77	1·110
NBSUA 3		·175					2·57	·757
Zagec	2½ × 2	—	·450	·175	·26	·18	—	—
Zagfe		·300					4·29	1·261
NBSUA 4		·250					4·04	1·188
Zaggi	3 × 2	·375	·500	·200	·27	·19	5·90	1·735
Zagho		·500					7·65	2·250
Zagid								
NBSUA 5		·250					4·46	1·313
Zagju	3 × 2½	·375	·525	·225	·29	·20	6·54	1·923
Zagof		·500					8·50	2·501
Zagug								
NBSUA 6		·250					4·89	1·438
Zahac	3½ × 2½	·375	·525	·250	·30	·21	7·17	2·110
Zahed		·500					9·35	2·750
Zahfa								

* See remarks at end of Table.

List 2

BRITISH STANDARD SECTIONS.

UNEQUAL ANGLES.

c_x c_y Distance of Centre of Gravity from back lines of Angle.

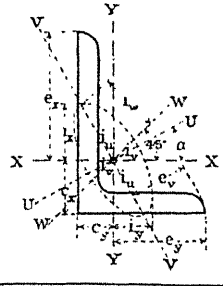
$J = \Sigma i^2$ Moment of Inertia.

$i = \sqrt{\frac{J}{a}}$ Radius of Gyration.

$\text{Tan. } 2\alpha = \frac{J_x + J_y - 2J_w}{J_y - J_x}$

e_x e_y e_v Distance of outer fibres from X, Y and V axes.

$Z_v = \frac{J_v}{e_v}$ Minimum Modulus of Section.



10	11	12	13	14	15	16	17	18	19	20	21	22	23
Centre of Gravity		Moments of Inertia.				Radii of Gyration.				Angle	Moduli of Section		Reference No
c_x	c_y	J_x	J_y	Max. J_u	Min. J_v	i_x	i_y	Max i_u	Min i_v	α	Z_x	Z_y	
inches		inches⁴.		inches⁴		inches		inches		tan α	inches³		
·621	·375	·224	·107	·273	·058	·620	·428	·685	·316	·542	·162	·095	NBSUA 1
·672	·425	·356	·168	·431	·094	·609	·419	·670	·313	·532	·268	·157	
·827	·337	·418	·112	·463	·068	·791	·410	·831	·319	·355	·250	·097	NBSUA 2
·882	·387	·673	·178	·741	·111	·779	·401	·817	·316	·347	·416	·160	
·740	·494	459	260	585	·134	·779	·586	·879	·421	·622	·261	·172	NBSUA 3
·793	·546	·743	·418	·943	·218	·768	·576	·865	·416	·617	·436	·288	
·977	·483	1·060	·375	1·218	·217	·945	·562	1·013	·427	·433	·524	·247	NBSUA 4
1·028	·532	1·505	·528	1·720	·311	·931	·551	·996	·424	·425	763	·359	
1·075	·579	1·896	·657	2·150	·403	·918	·540	·977	·423	·413	·985	·462	
·894	·647	1·139	·716	1·498	·357	·931	·738	1·088	·521	·678	·541	·386	NBSUA 5
·945	·697	1·623	1·015	2·126	·513	·919	·727	1·051	·516	·673	790	·663	
·992	·744	2·051	1·277	2·665	·663	·906	·714	1·032	·515	·665	1·021	·727	
1·094	·601	1·752	·745	2·084	·413	1·104	·720	1·204	·536	·498	·728	·392	NBSUA 6
1·146	·651	2·513	1·059	2·978	·594	1·091	·708	1·188	·531	·492	1·068	·573	
1·195	·698	3·193	1·335	3·761	·767	1·077	·697	1·169	·528	·484	1·385	·741	

List 2 (*cont.*)

BRITISH STANDARD SECTIONS

* UNEQUAL ANGLES.

(*continued*)

1	2	3	4	5	6 | 7	8	9
Reference No and Code Word	Size A × B	Standard Thickness t	Maximum Thickness Rolled	Minimum Thickness Rolled.	Radii r_1 | r_2	Mass per foot. W	Sectional Area a
	inches.	inches.	inches.	inches.	inches.	lb	inches
NBSUA 7		·250				5·32	1·564
Zahge	3½ × 3	·375	·550	·250	·32 ·22	7·81	2·298
Zahhi		·500				10·20	3·001
Zahif							
NBSUA 8		·300				6·84	2·011
Zahuh	4 × 3	·425	·575	·275	·33 ·23	9·50	2·795
Zaibe		·500				11·05	3·251
Zaici							
NBSUA 9		·300				7·35	2·162
Zaido	4 × 3½	·425	·600	·275	·35 ·24	10·23	3·008
Zaifu		·500				11·91	3·502
Zakaf							
NBSUA 10		·300				7·86	2·311
Zakeg	5 × 3	·425	·625	·300	·36 ·25	10·95	3·220
Zakha		·500				12·75	3·751
Zakih							
NBSUA 11		·375				10·37	3·049
Zakje	5 × 3½	—	·625	·325	·38 ·26	—	—
—		·500				13·61	4·002
Zakkl							
NBSUA 12		·375				11·00	3·236
Zaklo	5 × 4	—	·650	·325	·39 ·27	—	—
—		·500				14·45	4·251
Zakmu							
NBSUA 13		·375				11·00	3·236
Zakuk	6 × 3	—	·650	·325	·39 ·27	—	—
—		·500				14·45	4·251
Zalag							
NBSUA 14		·375				11·63	3·422
Zaleh	6 × 3½	—	·675	·350	·41 ·29	—	—
—		·500				15·30	4·500
Zalja							
NBSUA 15		·375				12·28	3·611
Zalke	6 × 4	—	·700	·350	·42 ·29	—	—
—		·500				16·16	4·752
Zalli							
NBSUA 16		—				—	—
—	7 × 3½	·525	·725	·375	·44 ·31	17·81	5·237
Zalmo		—				—	—
—							

*See remarks at end of Table

List 2 (*cont.*)

BRITISH STANDARD SECTIONS

UNEQUAL ANGLES.

(*continued*)

10	11	12	13	14	15	16	17	18	19	20	21	22	23
Centre of Gravity		Moments of Inertia				Radii of Gyration.				Angle	Moduli of Section.		Reference No.
c_x	c_y	J_x	J_y	Max. J_u	Min. J_v	i_x	i_y	Max. i_u	Min. i_v	α	Z_x	Z_y	
inches.		*inches⁴.*		*inches⁴.*		*inches*		*inches*		tan α	*inches³.*		
1·014	·768	1·856	1·256	2·503	·609	1·090	·896	1·265	·624	·720	·747	·563	NBSUA 7
1·067	·819	2·669	1·800	3·590	·878	1·078	·885	1·250	·618	·717	1·097	·825	
1·115	·867	3·399	2·283	4·547	1·135	1·064	·872	1·231	·615	·712	1·425	1·070	
1·235	·741	3·186	1·535	3·894	·825	1·259	·874	1·392	·641	·548	1·152	·679	NBSUA 8
1·286	·790	4·335	2·076	5·284	1·127	1·245	·862	1·375	·635	·544	1·598	·939	
1·315	·819	4·974	2·372	6·045	1·302	1·237	·854	1·364	·633	·540	1·853	1·088	
1·158	·911	3·343	2·385	4·595	1·133	1·244	1·050	1·458	·724	·763	1·176	·921	NBSUA 9
1·209	·961	4·560	3·243	6·252	1·551	1·231	1·038	1·441	·718	·750	1·634	1·278	
1·238	·990	5·236	3·718	7·162	1·793	1·223	1·030	1·430	·715	·748	1·896	1·481	
1·650	·662	5·910	1·617	6·553	·974	1·599	·837	1·684	·649	·381	1·764	·692	NBSUA 10
1·704	·713	8·100	2·195	8·962	1·332	1·586	·826	1·668	·643	·357	2·457	·960	
1·734	·742	9·331	2·513	10·305	1·538	1·577	·818	1·657	·640	·354	2·857	1·113	
1·589	·847	7·631	3·088	8·989	1·730	1·582	1·006	1·717	·753	·480	2·237	1·164	NBSUA 11
1·640	·896	9·842	3·955	11·560	2·237	1·568	·994	1·699	·748	·475	2·930	1·519	
1·507	1·012	7·967	4·532	10·178	2·321	1·569	1·183	1·774	·847	·626	2·281	1·517	NBSUA 12
1·558	1·061	10·289	5·828	13·111	3·007	1·556	1·171	1·756	·841	·622	2·989	1·983	
2·118	·633	11·991	2·049	12·722	1·317	1·925	·796	1·985	·638	·262	3·088	·866	NBSUA 13
2·171	·683	15·513	2·619	16·430	1·702	1·910	·785	1·966	·633	·258	4·052	1·130	
2·009	·772	12·617	3·212	13·874	1·956	1·920	·869	2·014	·756	·343	3·161	1·177	NBSUA 14
2·062	·822	16·355	4·127	17·950	2·532	1·906	·958	1·997	·750	·340	4·154	1·541	
1·914	·924	13·213	4·743	15·236	2·719	1·913	1·146	2·054	·868	·439	3·233	1·542	NBSUA 15
1·967	·975	17·142	6·113	19·731	3·524	1·899	1·134	2·038	·861	·436	4·250	2·020	
2·510	·774	26·191	4·447	27·764	2·875	2·236	·921	2·302	·741	·260	5·833	1·631	NBSUA 16

List 2 (*cont.*)

BRITISH STANDARD SECTIONS.

UNEQUAL ANGLES.

(*continued*)

1	2	3	4	5	6	7	8	9
Reference No. and Code Word	Size. A × B	Standard Thickness. t	Maximum Thickness Rolled.	Minimum Thickness Rolled.	Radii. Root. r_1	Radii. Toe. r_2	Calculated Weight per foot. W	Sectional Area. a
	inches.	inches.	inches.	inches.	inches		lb.	inches²
NBSUA 17 — Zainu —	8 × 4	·625 —	775	·425	·48	·34	24·17 —	7·109 —
NBSUA 18 — Zalok —	8 × 6	— ·625 —	·800	450	·54	·38	— 28·42 —	— 8 360 —
NBSUA 19 — Zalul —	9 × 4	— ·650 —	·825	·450	·51	36	— 27·30 —	— 8·028 —
NBSUA 20 — Zamah —	10 × 4	— ·675 —	·850	·475	·54	·38	— 30·58 —	— 8·995 —

Remarks.

— ... of Standard Unequal Angles shall be in accordance with ... h the angle between the flanges does ...

Angles ordered to the standard thickness shall be practically accurate in profile; but if the thickness is between, above ... the flanges will be proportionately longer or shorter than the Standards. ... Standards, instead of square; ... Sided Angles the flanges may ...

Angles may be ordered by width of flanges and thickness, or by width of flanges and weight per foot, but not by both thickness and weight per foot. Where thickness is employed in ordering, decimals of an inch shall be used. The Association suggests that all Angles be ordered by size of flanges and weight per foot.

List 2 (*cont.*)

BRITISH STANDARD SECTIONS.

UNEQUAL ANGLES.

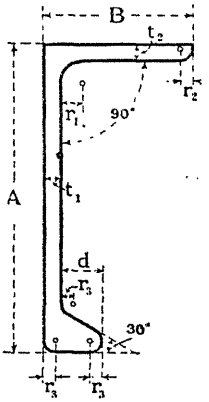
(*continued*)

10	11	12	13	14	15	16	17	18	19	20	21	22	23
Centre of Gravity.		Moments of Inertia.				Radii of Gyration.				Angle.	Moduli of Section.		Reference No.
c_x	c_y	J_x	J_y	Max. J_u	Min. J_v	i_x	i_y	Max. i_u	Min. i_v	α	Z_x	Z_y	
inches.		inches^4.		inches^4		inches.		inches.		tan α	inches^3.		
2·882	·895	46·376	7·871	49·150	5·096	2·554	1·052	2·629	·847	·259	9·060	2·535	NBSUA 17
2·492	1·500	53·266	25·741	65·211	13·795	2·524	1·755	2·793	1·285	·550	9·671	5·720	NBSUA 18
3·340	·857	66·521	8·328	69·275	5·575	2·879	1·019	2·938	·833	·213	11·763	2·650	NBSUA 19
3·806	·827	92·050	8·765	94·787	6·028	3·199	·987	3·246	·819	·178	14·860	2·762	NBSUA 20

List 3

BRITISH STANDARD SECTIONS.

* BULB ANGLES.



a = Sectional Area in square inches.

w = Weight in lb. per foot = 3·4 a

1	2	3	4	5	6	7	8	9	10	11	12
Reference No. and Code Word.	Size. A × B	Standard Thickness. Web. t_1	Standard Thickness. Flange. t_2	Maximum Web Thickness Rolled.	Minimum Web Thickness Rolled.	Radii. r_1	Radii. r_2	Radii. r_3	d	Calculated Weight per foot. w	Sectional Area. a
	inches	inches.		inches.	inches.	inches.			inches.	lb.	inches
NBSBA 1 Zamik	4 × 2½	·26		46	·21	·42	·21	·16	·52	6·29	1·849
NBSBA 2 Zamka	4½ × 2½	·28		·48	·23	·42	·21	·18	·58	7·34	2·158
NBSBA 3 Zamle	5 × 2½	·30		·50	·25	·42	·21	·20	·65	8·49	2·498
NBSBA 4 Zammi	5 × 3	·30		·50	·25	·48	·24	·20	·65	9·03	2·657
NBSBA 5 Zamno	5½ × 3	·31		·52	·26	·48	·24	·22	·71	10·02	2·947
NBSBA 6 Zamol	6 × 3	·33		·54	·28	·48	·24	·24	·78	11·37	3·344
NBSBA 7 Zampu	6 × 3½	·33		·54	·28	·54	·27	·24	·78	11·96	3·519
NBSBA 8 Zamum	7 × 3	·38		·59	·33	·48	·24	·28	·91	14·62	4·300

*See remarks at end of Table.

List 3

BRITISH STANDARD SECTIONS.

BULB ANGLES.

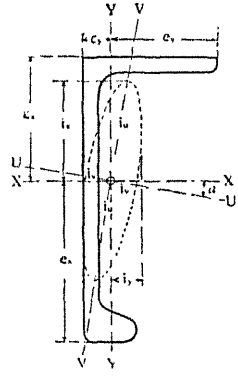
c_x c_y Distance of Centre of Gravity from back lines of Angle

$J = ai^2$ Moment of Inertia.

$i = \sqrt{\frac{J}{a}}$ Radius of Gyration.

e_x e_y Distance of outer fibres from X and Y axes

$Z = \frac{J}{e}$ Modulus of Section.



13	14	15	16	17	18	19	20	21	22	23	24	25	26
Centre of Gravity		Moments of Inertia				Radii of Gyration				Angle.	Moduli of Section.		Reference No
c_x	c_y	J_x	J_y	Max J_u	Min J_v	i_x	i_y	Max i_u	Min i_v	α	Z_x	Z_y	
inches		inches^4		inches^4		inches		inches		tan α	inches^3		
1·577	·555	3·798	·805	4 036	·568	1·433	·660	1·477	·554	·271	1·568	·414	NBSBA 1
1·848	·540	5·641	·886	5·869	·657	1·617	641	1·649	·552	·214	2·127	452	NBSBA 2
2·131	·529	8·094	·968	8·308	·753	1·800	·622	1·824	·549	·171	2·821	·491	NBSBA 3
2·014	·650	8·669	1·635	9·128	1·176	1·806	·784	1·854	·666	·247	2·904	·696	NBSBA 4
2 295	·631	11·733	1·724	12·164	1·293	1·895	·765	2·032	·662	·203	3·661	·728	NBSBA 5
2·581	·622	15·865	1·866	16·277	1·454	2·178	·747	2·206	·659	·169	4·640	·785	NBSBA 6
2·462	·740	16·819	2·903	17·603	2·118	2·186	·908	2·237	·776	·231	4·754	1·052	NBSBA 7
3·147	·616	27·480	2·213	27·856	1·837	2·628	·717	2·645	·654	·121	7·133	·928	NBSBA 8

List 3 (*cont.*)

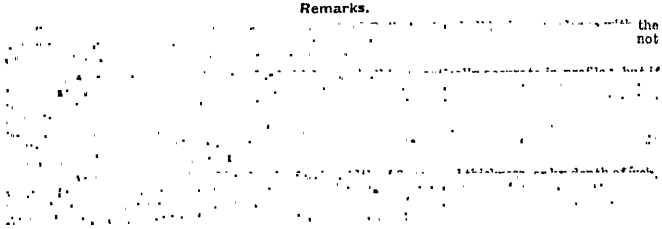
BRITISH STANDARD SECTIONS.

BULB ANGLES.

(*continued*)

1	2	3	4	5	6	7	8	9	10	11	12
Reference No and Code Word.	Size A × B	Standard Thickness Web. t_1	Standard Thickness Flange. t_2	Maximum Web Thickness Rolled.	Minimum Web Thickness Rolled.	Radii r_1	Radii r_2	Radii r_3	d	Calculated Weight per foot. w	Sectional Area a
	inches.	inches.		inches.	inches	inches.			inches	lb	inches^2
NBSBA 9 Zanek	7 × 3½	·38		·59	·33	·54	·27	·28	·91	15·30	4·500
NBSBA 10 Zanil	8 × 3	·40		·61	·35	·48	·24	·32	1·04	17·24	5·070
NBSBA 11 Zania	8 × 3½	·40		·61	·35	·54	·27	·32	1·04	17·95	5·280
NBSBA 12 Zanme	9 × 3½	·43		·65	·38	·54	·27	·36	1·17	21·22	6·241
NBSBA 13 Zanni	10 × 3½	·45		·67	·40	·54	·27	·40	1·30	24·34	7·159
NBSBA 14 Zanom	11 × 3½	·48		·70	·43	·54	·27	·44	1·43	28·14	8·276
NBSBA 15 Zanpo	12 × 3½	·50		·73	·45	·54	·27	·48	1·56	31 73	9·332
NBSBA 16 Zanru	12 × 4	·50		·73	·45	·60	30	·48	1·56	32·62	9·593
NBSBA 17 Zanum	13½ × 4	·54		·77	·49	·60	·30	·54	1·75	38·98	11·466
NBSBA 18 Zapak	15 × 4	·57		·82	·52	·60	·30	·60	1·95	45·40	13·352

Remarks.



List 3 (*cont.*)

BRITISH STANDARD SECTIONS.

BULB ANGLES.

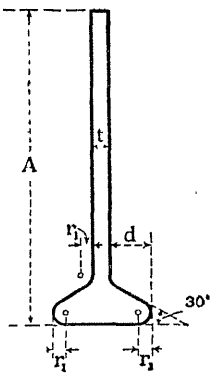
(*continued*)

13	14	15	16	17	18	19	20	21	22	23	24	25	26
Centre of Gravity.		Moments of Inertia.				Radii of Gyration.				Angle	Moduli of Section.		Reference No.
c_x	c_y	J_x	J_y	Max J_u	Min J_v	i_x	i_y	Max i_u	Min. i_v	α	Z_x	Z_y	
inches		inches⁴.		inches⁴.		inches		inches.		tan α	inches³.		
3·017	·724	29·130	3·425	29·887	2·668	2·544	·872	2·577	·770	·169	7·314	1·234	NBSBA 9
3·752	·607	42·303	2·436	42·603	2·136	2·888	·693	2·899	·649	·086	9·957	1 018	NBSBA 10
3·611	·704	44·819	3·729	45·472	3·076	2·913	·840	2·935	·763	·125	10·212	1·334	NBSBA 11
4·208	·697	66·703	4·147	67·261	3·589	3·269	·815	3·283	·768	·094	13·919	1·480	NBSBA 12
4·825	·694	94·102	4·526	94 545	4·083	3·626	·795	3·634	·755	·070	18·185	1·613	NBSBA 13
5·437	·700	130·469	5·054	130·808	4·715	3·970	·781	3·976	·755	·052	23·463	1·805	NBSBA 14
6·069	·707	173·935	5·570	174·166	5·339	4·317	·773	4·320	·756	·037	29·326	1·994	NBSBA 15
5·911	·784	182·473	7·750	183·096	7·127	4·361	·899	4·369	·862	·060	29·968	2·410	NBSBA 16
6·841	·795	272·631	8·947	273·037	8·540	4·876	·883	4·880	·863	·039	40·941	2·792	NBSBA 17
7·807	812	388·085	10·289	388·285	10·089	5·391	·878	5·393	·869	·023	53·949	3·228	NBSBA 18

List 4

BRITISH STANDARD SECTIONS.

BULB PLATES.



a = Sectional Area in square inches.

w = Weight in lb. per foot = 3·4 a

1	2	3	4	5	6	7	8	9
Reference No. and Code Word.	Size. A	Standard Thickness. t	Maximum Thickness Rolled.	Minimum Thickness Rolled.	Radius. r_1	d	Calculated Weight per foot. w	Sectional Area. a
	inches.	inches.	inches.	inches.	inches.	inches.	lb.	inches^2.
NBSBP 1 Zapel	7	·35	·45	·275	·28	·91	12·57	3·698
NBSBP 2 Zapim	10	·50	·625	·375	·40	1·30	25·66	7·547

Remarks.

The depth of web, thickness and profile of Standard Bulb Plates shall be in accordance with the accompanying list and sketch.

Bulb Plates ordered to the standard thickness shall be practically accurate in profile, but if the bulb

of web and weight per
employed in ordering
Plates be ordered by

List 4

BRITISH STANDARD SECTIONS.

BULB PLATES.

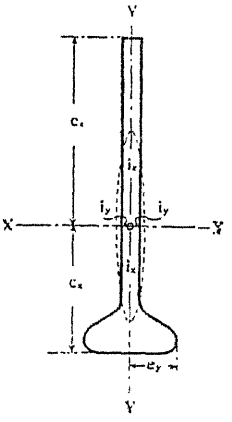
c_x c_y Distance of Centre of Gravity from bottom line and Y axis.

$J = ai^2$ Moment of Inertia.

$i = \sqrt{\frac{J}{a}}$ Radius of Gyration.

e_x e_y Distance of outer fibres from X and Y axes.

$Z = \frac{J}{e}$ Modulus of Section.

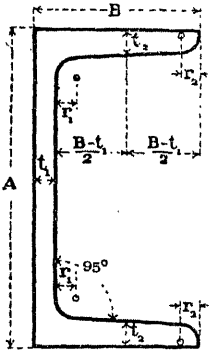


10	11	12	13	14	15	16	17	18
Centre of Gravity.		Moments of Inertia.		Radii of Gyration.		Moduli of Section.		Reference No.
c_x	c_y	J_x	J_y	i_x	i_y	Z_x	Z_y	
inches.		inches^4.		inches.		inches^3		
2·447	0	18·118	·487	2·213	·363	3·380	·448	NBSBP 1
3·496	0	75·462	2·027	3·162	·518	11·602	1·308	NBSBP 2

List 5

BRITISH STANDARD SECTIONS.

*CHANNELS.



a = Sectional Area in square inches.

W = Weight in lb. per foot = 3·4 a

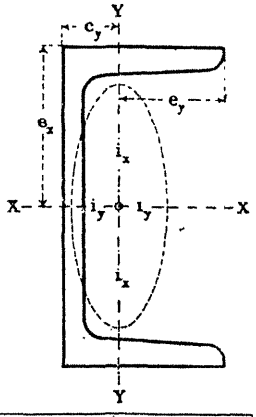
1	2	3	4	5	6	7	8
Reference No and Code Word.	Size.	Standard (Minimum) Thickness.		Radii		Calculated Weight per foot.	Sectional Area
	A × B	Web t_1	Flange t_2	Root. r_1	Toe r_2	W	a
	inches.	inches.		inches.		lb.	inches².
NBSC 1 Zapma	3 × 1½	·20	·28	·30	·15	4·60	1·352
NBSC 2 Zapne	4 × 2	·24	·31	·36	·18	7·09	2·085
NBSC 3 Zapon	5 × 2½	·25	·38	·42	·21	10·22	3·006
NBSC 4 Zappl	6 × 3	·25	·38	·48	·24	12·41	3·650
NBSC 5 Zapro	6 × 3½	·28	·48	·54	·27	16·48	4·848

*See remarks at end of Table.

List 5

BRITISH STANDARD SECTIONS.

CHANNELS.



c_x c_y Distance of Centre of Gravity from X axis and back line of Channel

$J = ai^2$ Moment of Inertia

$i = \sqrt{\frac{J}{a}}$ Radius of Gyration

e_x e_y Distance of outer fibres from X and Y axes

$Z = \frac{J}{e}$ Modulus of Section

9	10	11	12	13	14	15	16	17
Centre of Gravity		Moments of Inertia		Radii of Gyration.		Moduli of Section		Reference No.
c_x	c_y	J_x	J_y	i_x	i_y	Z_x	Z_y	
inches.		inches^4		inches.		inches^3.		
0	·476	1·823	·261	1·161	·439	1·215	·255	NBSC 1
0	·599	5·063	·703	1·558	·581	2·532	·502	NBSC 2
0	·773	11·873	1·641	1·987	·739	4·749	·950	NBSC 3
0	·890	21·271	2·825	2·414	·880	7·090	1·339	NBSC 4
0	1·143	29·882	5·293	2·441	1·045	9·627	2·246	NBSC 5

List 5 (*cont.*)

BRITISH STANDARD SECTIONS.

*CHANNELS.

(*continued*)

1	2	3	4	5	6	7	8
Reference No. and Code Word.	Size. A × B	Web. t_1	Flange. t_2	Root. r_1	Toe. r_2	Calculated Weight *per foot.* W	Sectional Area. a
	inches.	inches.		inches.		lb.	inches^2.
NBSC 6 Zapsu	7 × 3	·26	·42	·48	·24	14·22	4·182
NBSC 7 Zapup	7 × 3½	·30	·50	·54	·27	18·28	5·376
NBSC 8 Zaral	8 × 3	·28	·44	·48	·24	15·96	4·694
NBSC 9 Zarem	8 × 3½	·32	·52	·54	·27	20·21	5·944
NBSC 10 Zarin	9 × 3	·30	·44	·48	·24	17·46	5·136
NBSC 11 Zarna	9 × 3½	·34	·54	·54	·27	22·27	6·549
NBSC 12 Zarop	10 × 3	·32	·45	·48	·24	19·28	5·672
NBSC 13 Zarpe	10 × 3½	·36	·56	·54	·27	24·46	7·193
NBSC 14 Zarri	12 × 3½	·35	·50	·54	·27	25·25	7·426
NBSC 15 Zarso	12 × 3½	·40	·60	·54	·27	29·23	8·596
NBSC 16 Zartu	12 × 4	·40	·60	·60	·30	31·33	9·214

*See remarks at end of Table.

List 5 (*cont.*)

BRITISH STANDARD SECTIONS.

CHANNELS.

(*continued*)

9	10	11	12	13	14	15	16	17
Centre of Gravity.		Moments of Inertia.		Radii of Gyration.		Moduli of Section.		Reference No.
c_x	c_y	J_x	J_y	i_x	i_y	Z_x	Z_y	
inches.		inches^4		inches.		inches^3		
0	·875	32·750	5·255	2·798	·882	9·357	1·531	NBSC 6
0	1·092	42·834	5·834	2·823	1·042	12·238	2·423	NBSC 7
0	·834	46·720	3·578	3·155	·873	11·680	1·652	NBSC 8
0	1·045	60·573	6·370	3·192	1·035	15·143	2·595	NBSC 9
0	·781	62·522	3·752	3·489	·855	13·894	1·691	NBSC 10
0	1·003	82·617	6·899	3·552	1·026	18·359	2·763	NBSC 11
0	·742	82·663	3·983	3·817	·838	16·533	1·764	NBSC 12
0	·965	109·520	7·420	3·902	1·018	21·804	2·927	NBSC 13
0	·849	156·390	7·066	4·589	·976	26·065	2·685	NBSC 14
0	·901	180·292	8·436	4·590	·991	30·049	3·245	NBSC 15
0	1·055	200·086	12·120	4·660	1·147	33·348	4·116	NBSC 16

List 5 (*cont.*)

BRITISH STANDARD SECTIONS.

CHANNELS.

(*continued*)

1	2	3	4	5	6	7	8
Reference No. and Code Word.	Size.	Standard (Minimum) Thickness.		Radii.		Calculated Weight per foot.	Sectional Area.
		Web.	Flange.	Root.	Toe.		
	A × B	t_1	t_2	r_1	r_2	w	a
	inches.	inches.		inches.		lb.	inches^2.
NBSC 17 Zaruv	15 × 4	·41	·62	·60	·30	36·37	10·696
NBSC 18 Zasam	17 × 4	·48	·68	·60	·30	44·34	13·041

Remarks.

[illegible] Standard Channels shall be in accordance with the [illegible] measured at distances half-[illegible]

Channels ord[illegible] profile; but if the thickness is greater [illegible] the flanges will be increased by the sam[illegible]

Channels may be ordered by depth and thickness of web and width of flanges, or by size of web and flanges and weight per foot, but not by both thickness and weight per foot. Where thickness is employed in ordering, decimals of an inch shall be used. The Association suggests that all Channels be ordered by size of web and flanges and weight per foot.

List 5 (*cont.*)

BRITISH STANDARD SECTIONS.

CHANNELS.

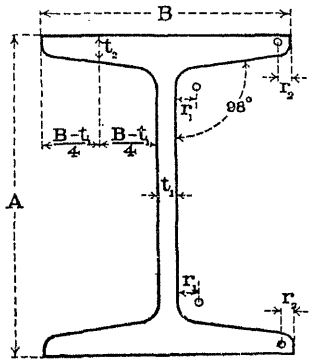
(*continued*)

9	10	11	12	13	14	15	16	17
Centre of Gravity.		Moments of Inertia.		Radii of Gyration.		Moduli of Section.		Reference No.
c_x	c_y	J_x	J_y	i_x	i_y	Z_x	Z_y	
inches.		inches^4.		inches.		inches^3.		
0	·967	349·095	13·338	5·713	1·117	46·546	4·398	NBSC 17
0	·920	520·178	15·263	6·316	1·082	61·197	4·955	NBSC 18

List 6

BRITISH STANDARD SECTIONS.

*BEAMS.



a = Sectional Area in square inches.

3·4 a = Weight in lb. per foot (approximately)

1	2	3	4	5	6	7	8
Reference No. and Code Word.	Size. A × B	Approximate Weight per foot.	Standard Thickness. Web. t_1	Standard Thickness. Flange. t_2	Radii. Root. r_1	Radii. Toe. r_2	Sectional Area. a
	inches.	lb.	inches.		inches.		inches²
	A. GIRDER SECTIONS.						
NBSB 1 Zasen	3 × 1½	4	·16	·249	·25	·12	1·177
NBSB 2 Zasor	4 × 1¾	5	·17	·239	·27	·13	1·470
NBSB 3 Zaspa	4½ × 2	7	·19	·322	·29	·14	2·060
NBSB 4 Zassi	5 × 2½	9	·20	·347	·33	·16	2·647
NBSB 5 Zasto	6 × 3	12	·23	·377	·37	·18	3·533
NBSB 6 Zasus	7 × 3½	15	·25	·398	·41	·20	4·416

*See remarks at end of Table.

List 6

BRITISH STANDARD SECTIONS.

BEAMS.

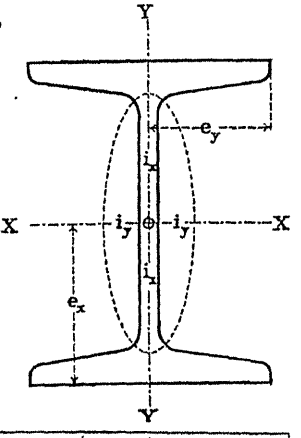
c_x c_y Distance of Centre of Gravity from X axis and Y axis.

$J = ai^2$ Moment of Inertia.

$i = \sqrt{\frac{J}{a}}$ Radius of Gyration.

e_x e_y Distance of outer fibres from X and Y axes.

$Z = \frac{J}{e}$ Modulus of Section.



9	10	11	12	13	14	15	16	17
Centre of Gravity		Moments of Inertia.		Radii of Gyration.		Moduli of Section.		Reference No.
c_x	c_y	J_x	J_y	i_x	i_y	Z_x	Z_y	
inches.		inches^4		inches.		inches^3.		
A. GIRDER SECTIONS.								
0	0	1·660	·125	1·188	·326	1·107	·167	NBSB 1
0	0	3·664	·186	1·579	·356	1·832	·213	NBSB 2
0	0	6·652	·383	1·797	·431	2·957	·383	NBSB 3
0	0	10·910	·789	2·030	·546	4·364	·631	NBSB 4
0	0	20·989	1·461	2·437	·643	6·996	·974	NBSB 5
0	0	35·904	2·408	2·851	·738	10·258	1·376	NBSB 6

List .6 (*cont.*)

BRITISH STANDARD SECTIONS.

*BEAMS.

(*continued*)

1	2	3	4	5	6	7	8
Reference No. and Code Word.	Size. A × B	Approximate Weight per foot.	Standard Thickness. Web. t_1	Flange. t_2	Radii. Root. r_1	Toe. r_2	Sectional Area. a
	inches.	lb.	inches.		inches		inches^2
NBSB 7 Zatan	8 × 4	18	·28	·398	·45	·22	5·296
NBSB 8 Zatep	9 × 4	21	·30	·457	·45	·22	6·177
NBSB 9 Zatir	10 × 4½	25	·30	·505	·49	·24	7·354
NBSB 10 Zatos	12 × 5	30	·33	·507	·53	·26	8·827
NBSB 11 Zatra	13 × 5	35	·35	·604	·53	·26	10·298
NBSB 12 Zatse	14 × 5½	40	·37	·627	·57	·28	11·765
NBSB 13 Zatti	15 × 6	45	·38	·655	·61	·30	13·236
NBSB 14 Zatut	16 × 6	50	·40	·726	·61	·30	14·705
NBSB 15 Zatvo	18 × 6	55	·42	·757	·61	·30	16·182
NBSB 16 Zauca	20 × 6½	65	·45	·820	·65	·32	19·119
NBSB 17 Zaude	22 × 7	75	·50	·834	·69	·34	22·064
NBSB 18 Zaufi	24 × 7½	90	·52	·984	·73	·36	26·465

* See remarks at end of Table.

List 6 (*cont.*)

BRITISH STANDARD SECTIONS.

BEAMS.

(*continued*)

9	10	11	12	13	14	15	16	17
Centre of Gravity.		Moments of Inertia.		Radii of Gyration.		Moduli of Section.		Reference No
c_x	c_y	J_x	J_y	i_x	i_y	Z_x	Z_y	
inches.		inches^4		inches.		inches^3		
0	0	55·629	3·506	3·241	·814	13·907	1·753	NBSB 7
0	0	81·127	4·148	3·624	·820	18·028	2·074	NBSB 8
0	0	122·338	6·486	4·079	·939	24·468	2·883	NBSB 9
0	0	206·931	8·770	4·842	·997	34·488	3·508	NBSB 10
0	0	283·507	10·815	5·247	1·025	43·616	4·326	NBSB 11
0	0	377·059	14·788	5·661	1·121	53·866	5·377	NBSB 12
0	0	491·912	19·871	6·096	1·225	65·588	6·624	NBSB 13
0	0	618·092	22·468	6·483	1·236	77·261	7·489	NBSB 14
0	0	841·759	23·635	7·212	1·209	93·529	7·878	NBSB 15
0	0	1226·172	32·559	8·008	1·305	122·617	10·018	NBSB 16
0	0	1670·796	41·055	8·718	1·364	152·436	11·733	NBSB 17
0	0	2443·227	60·436	9·608	1·511	203·602	16·116	NBSB 18

List 6 (*cont.*)

BRITISH STANDARD SECTIONS.

BEAMS.

(*continued*)

1	2	3	4	5	6	7	8
Reference No. and Code Word	Size. A × B	Approximate Weight per foot.	Standard Thickness. Web. t_1	Flange. t_2	Radii. Root. r_1	Toe. r_2	Sectional Area. a
	inches.	lb	inches.		inches		inches²
	B. HEAVY BEAMS AND PILLARS.						
NBSHB 1 Zaugo	4 × 3	10	·24	·347	·37	·18	2·940
NBSHB 2 Zavap	5 × 4½	20	·29	·513	·49	·24	5·882
NBSHB 3 Zaver	6 × 5	25	·33	·561	·53	·26	7·351
NBSHB 4 Zavis	8 × 6	35	·35	·648	·61	·30	10·296
NBSHB 5 Zavot	9 × 7	50	·40	·825	·69	·34	14·712
NBSHB 6 Zavvi	10 × 6	40	·36	709	·61	·30	11·771
NBSHB 7 Zawar	10 × 8	55	·40	·783	·77	·38	16·177
NBSHB 8 Zawes	12 × 8	65	·43	·904	·77	·38	19·122
NBSHB 9 Zawit	14 × 8	70	·46	·920	·77	·38	20·589
NBSHB 10 Zawta	16 × 8	75	·48	·938	·77	·38	22·063
NBSHB 11 Zawyu	18 × 8	80	·50	·950	·77	·38	23·526

Remarks.

[illegible] and profile of Standard Beams shall be in accordance with the accompanying list [illegible] all be measured at distances half-way between the extreme

[illegible] be practically accurate in profile: but if the thickness of the [illegible]lth of the section will be decreased or increased by the same

Beams may be ordered by depth of section, width of flanges and thickness, or by depth of section, width of flanges and weight per foot, but not by both thickness and weight per foot. Where thickness is employed in ordering, decimals of an inch shall be used. The Association suggests that all Beams be ordered by depth of section, width of flanges and weight per foot.

List 6 (*cont.*)

BRITISH STANDARD SECTIONS.

BEAMS.

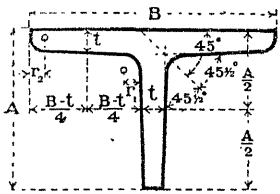
(*continued*)

9	10	11	12	13	14	15	16	17
Centre of Gravity.		Moments of Inertia.		Radii of Gyration		Moduli of Section		Reference No.
c_x	c_y	J_x	J_y	i_x	i_y	Z_x	Z_y	
inches		inches^4		inches.		inches^3.		
B.		**HEAVY BEAMS AND**				**PILLARS.**		
0	0	7·788	1·326	1·627	·672	3·893	·884	NBSHB 1
0	0	25·025	6·590	2·063	1·058	10·010	2·929	NBSHB 2
0	0	45·162	9·876	2·479	1·159	15·054	3·951	NBSHB 3
0	0	115·058	19·540	3·343	1·378	28·764	6·513	NBSHB 4
0	0	208·128	40·169	3·761	1·652	46·251	11·477	NBSHB 5
0	0	204·803	21·759	4·171	1·360	40·961	7·253	NBSHB 6
0	0	298·683	54·743	4·224	1·840	57·738	13·686	NBSHB 7
0	0	407·769	65·184	5·051	1·846	81·295	16·296	NBSHB 8
0	0	705·575	66·674	5·854	1·800	100·796	16·668	NBSHB 9
0	0	973·909	88·303	6·644	1·759	121·739	17·076	NBSHB 10
0	0	1292·073	69·427·	7·411	1·718	143·564	17·357	NBSHB 11

List 7

BRITISH STANDARD SECTIONS.

*T BARS.



a = Sectional Area in square inches.

w = Weight in lb. per foot = 3·4 a

1	2	3	4	5	6	7
Reference No. and Code Word.	Size. Flange. Web. B × A	Standard Thickness. t	Radii. Root. r_1	Toe. r_2	Calculated Weight per foot. w	Sectional Area. a
	inches.	inches.	inches.		lb.	inches².
NBST 1		·125			·82	·240
Zaxas	1 × 1	·187	·18	·13	1·17	·344
Zaxet		—			—	—
—						
NBST 2		·187			1·81	·533
Zaxow	1½ × 1½	·250	·21	·15	2·36	·693
Zaxux		—			—	—
—						
NBST 3		·250			3·21	·945
Zaxva	2 × 2	·312	·24	·17	3·94	1·158
Zaxwe		·375			4·64	1·366
Zaxxi						
NBST 4		·250			4·07	1·197
Zaxzu	2½ × 2½	·312	·27	·19	5·00	1·471
Zayat		·375			5·92	1·742
Zaybu						

*See remarks at end of Table.

List 7

BRITISH STANDARD SECTIONS.

T BARS.

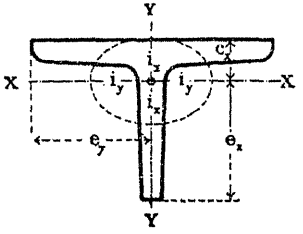
c_x c_y Distance of Centre of Gravity from top line and Y axis.

$J = ai^2$ Moment of Inertia.

$i = \sqrt{\frac{J}{a}}$ Radius of Gyration.

e_x e_y Distance of outer fibres from X and Y axes.

$Z = \frac{J}{e}$ Modulus of Section



8	9	10	11	12	13	14	15	16
Centre of Gravity		Moments of Inertia.		Radii of Gyration.		Moduli of Section.		Reference No.
c_x	c_y	J_x	J_y	i_x	i_y	Z_x	Z_y	
inches.		inches^4		inches.		inches^3.		
·289	0	·021	·009	·256	·195	·030	·018	NBST 1
·314	0	·029	·015	·291	·206	·043	·029	
·434	0	·106	·048	·447	·302	·100	·065	NBST 2
·459	0	·135	·067	·442	·312	·130	·090	
·579	0	·337	·157	·597	·408	·237	·157	NBST 3
·604	0	·406	·201	·592	·417	·291	·201	
·628	0	·469	·246	·586	·425	·342	·246	
·697	0	·678	·305	·752	·504	·378	·244	NBST 4
·724	0	·822	·388	·748	·514	·463	·311	
·749	0	·959	·475	·742	·522	·548	·380	

List 7 (*cont.*)

BRITISH STANDARD SECTIONS.

T BARS.

(*continued*)

1	2	3	4	5	6	7
Reference No. and Code Word.	Size. Flange. B × Web. A	Standard Thickness. t	Radii. Root. r_1	Radii. Toe. r_2	Calculated Weight per foot. W	Sectional Area. a
	inches.	inches.	inches.		lb.	inches².
NBST 5 Zayox Zaywa Zazbo	3 × 3	·312 ·375 ·437	·30	·21	6·07 7·20 8·30	1·786 2·119 2·440
NBST 6 Zazcu Zazix —	4 × 3	·375 ·500 —	·33	·23	8·49 11·09 —	2·498 3·262 —
NBST 7 Zazoy Zazzi —	5 × 3	·375 ·500 —	·36	·25	9·79 12·80 —	2·878 3·766 —
NBST 8 Zebax Zebbe —	5 × 4	·500 ·625 —	·39	·27	14·50 17·84 —	4·266 5·248 —
NBST 9 Zebcl Zebdo —	6 × 4	·500 ·625 —	·42	·29	16·22 19·99 —	4·772 5·878 —
NBST 10 Zebey Zebfu —	6 × 6	·500 ·625 —	·48	·34	19·62 24·23 —	5·772 7·126 —

Remarks.

[illegible] web.

Tees ordered to the standard thickness shall be practically accurate in profile.

[illegible] Where thickness [illegible] sts that all Tees be

The tapers of the flange and web to be such that the under side of the flange form an angle of ½° with the horizontal upper side, whilst each side of the web form an angle of ½° with the vertical centre line as shown in the diagram on page 58.

List 7 (*cont.*)

BRITISH STANDARD SECTIONS.

T BARS.

(*continued*)

8	9	10	11	12	13	14	15	16
Centre of Gravity.		Moments of Inertia.		Radii of Gyration.		Moduli of Section.		Reference No.
c_x	c_y	J_x	J_y	i_x	i_y	Z_x	Z_y	
inches.		inches^4.		inches.		inches^3		
·842	0	1·455	·666	·801	·611	·674	·444	NBST 5
·869	0	1·708	·813	·898	·619	·801	·542	
·894	0	1·942	·959	·892	·627	·922	·640	
·766	0	1·860	1·911	·863	·875	·833	·956	NBST 6
·816	0	2·366	2·595	·852	·892	1·083	1·298	
·691	0	1·973	3·716	·828	1·136	·855	1·487	NBST 7
·741	0	2·515	5·037	·817	1·157	1·113	2·015	
1·052	0	5·773	5·024	1·163	1·085	1·958	2·009	NBST 8
1·102	0	6·963	6·363	1·152	1·101	2·403	2·545	
·968	0	6·070	8·642	1·128	1·346	2·002	2·881	NBST 9
1·018	0	7·331	10·932	1·117	1·364	2·469	3·644	
1·630	0	19·043	8·556	1·816	1·218	4·357	2·852	NBST 10
1·687	0	23·309	10·866	1·809	1·235	5·404	3·622	

CHAPTER XX

FIRE-RESISTING CONSTRUCTION

Definition.—Buildings and all parts of buildings designed and arranged to retard the action of fire are termed fire-resisting constructions. Reinforced concrete constructions are given in a separate chapter.

Generally.—Fire-resisting constructions have been the subject of much study during recent years, and considerable advances in this direction have been made in all modern buildings of any importance. The experience obtained from all the great fires of late years, however, leaves no doubt that the construction of a furnace would be necessary to withstand the intense heat of a general conflagration. The whole effort of the designer must, therefore, be to select fire-resisting materials from those at his disposal, and to arrange them to comply with the modern requirements. Up to the present time no building has been erected perfectly fire-proof, though much has been done to render them fire-resisting.

Fire-resisting floors, among their other advantages, are eminently sanitary, and are, therefore, suitable for public buildings, notably hospitals, and are noiseless if covered with a suitable covering (see "Elementary Course").

Bye-laws affecting Fire-resisting Materials.—The London County Council Bye-laws give the following definition :—

"Incombustible material" means a material which neither burns nor gives off inflammable vapour in sufficient quantity to ignite when heated in the manner specified in the British Standard Specification (Definitions for Fire Resistance, Incombustibility and non-Inflammability of Building Materials and Structures), numbered 476.

No schedule of materials having fire-resisting properties is given in the L.C.C. Bye-laws or in the above B.S.S. No. 476, which contains definitions and methods of test only. The practice of specifying materials which have by experience proved suitable will no doubt be revived. The following schedule, now repealed, formed part of the London Building Acts for many years and can be taken as a guide until new legal provisions are stated based on the B.S.S. requirements.

(I) For general purposes :

(1) Brickwork constructed of good bricks well burnt hard and sound properly bonded and solidly put together—

(*a*) With good mortar compounded of good lime and sharp clean sand hard clean broken brick broken flint grit or slag ; or

(*b*) With good cement ; or

(*c*) With cement mixed with sharp clean sand hard clean broken brick broken flint grit or slag ;

(2) Granite and other stone suitable for building purposes by reason of its solidity and durability :

(3) Iron steel and copper ;

(4) Slate tiles brick and terra-cotta when used for coverings or corbels ;

(5) Flagstones when used for floors over arches but such flagstones not to be exposed on the underside and not supported at the ends only ;

(6) Concrete composed of broken brick tile stone chippings ballast pumice or coke breeze and lime cement or calcined gypsum ;

(7) Any combination of concrete and steel or iron.

(II) For special purposes :

(1) In the case of doors and shutters and their frames oak teak jarrah karri or other hard timber not less than one and three-quarters inches finished thickness the frames being bedded solid to the walls or partitions ;

(2) In the case of staircases and landings oak teak jarrah karri or other hard timber the treads risers strings and bearers being not less than one and three-quarters inches finished thickness and the ceilings and soffits if any being of plaster or cement.

(3) Oak teak jarrah karri and other hard timber when

used for beams or posts or in combination with iron the timber and the iron (if any) being protected by plastering or other incombustible or non-conducting external coating not less than two inches in thickness or in the case of timber not less than one inch in thickness on iron lathing ;

(4) (*a*) In the case of floors and roofs—

Brick tile terra-cotta or concrete composed as described in paragraph (I) (6) of this schedule not less than five inches thick in combination with iron or steel ;

(*b*) In the case of floors and of the roofs of projecting shops—

Pugging of concrete composed as described in the said paragraph (I) (6) not less than five inches thick, between wood joists provided a fillet one inch square is secured to the sides of the joists and placed so as to be in a central position in the depth of the concrete or concrete blocks not less than five inches thick laid between wood joists on fire-resisting bearers secured to the sides of the joists ;

(5) In the case of verandahs balustrades outside landings the treads strings and risers of outside stairs outside steps porticoes and porches oak teak jarrah karri or other hard timber not less than one and three-quarters inches finished thickness ;

(6) In the case of internal partitions enclosing staircases and passages terra-cotta brickwork concrete or other incombustible material not less than three inches thick ;

(7) In the case of glazing for windows doors and borrowed lights lantern or skylights glass not less than one fourth of an inch in thickness in direct combination with metal the melting point of which is not lower than 1,800 degrees Fahrenheit in squares not exceeding sixteen square inches and in panels not exceeding two feet across either way the panels to be secured with fire-resisting materials in fire-resisting frames of hard wood not less than one and three-quarters inches finished thickness or of iron.

(III) Any other material from time to time approved by the Council as fire-resisting.

Materials.—The necessary characteristics of a typical fire-resisting material are as follows :—1st, It should not

consume or become disintegrated under great heat; 2nd, its expansion, when heated, should not be sufficient to damage the structure of which it forms a part; 3rd, its contraction, when heated to a high temperature and then suddenly cooled with water, should not be sufficiently rapid as to cause it to fly to pieces. Although none of the materials in use possess the whole of the above qualities, many of them resist the action of fire to an extent which renders them valuable for fire-resisting work, and it only remains to adapt them to the requirements of the structure. The materials chiefly used, and their fitness for the four principal parts of any structure, viz., walls, floors, staircases, and roofs, will be described.

Stone.—Stone is a bad material to resist the action of fire, for when heated and suddenly cooled with water it is liable to fly to pieces, and it is useful only because it is non-combustible. Granite, when subjected to a great heat, crumbles to a fine sand, or cracks and falls to pieces with a series of small explosions. Limestones are calcined and turned to quick lime, the particles of which lose all cohesion when exposed to air and water. Sandstones resist fire better than the previous two, but after a short exposure they become disintegrated.

Bricks.—Bricks form one of the best materials for the construction of walls, which, if well-flushed with good mortar, and of an uninterrupted thickness of at least 9 inches, are practically fireproof; but alone for floors, roofs, and stairs they cannot be economically adopted, as it would be necessary to construct arches, which exert great thrust on the walls, rendering iron ties or thick walls necessary; this is expensive and is a barrier to their use.

Firebricks, from their refractory character, are best; but their comparatively great cost excludes their use for all ordinary works.

Terra-cotta.—Terra-cotta is a similar material to brick, its refractory qualities being superior to ordinary bricks, and it is useful for all purposes where brick is used; its expense, however, prevents its use for any but ornamental work. It is chiefly employed in fire-resisting constructions to protect iron girders, columns, and stanchions.

Numerous specially shaped fire-resisting, hollow, terra-cotta fire-brick, or brick-earth voussoirs are made by different firms for the construction of fire-resisting floors, with steel joists spaced about 2 feet apart, fire-resisting voussoirs and concrete, and obviating the necessity of timber centering.

Cast Iron.—Cast iron when heated and suddenly cooled by water usually flies into fragments; it is, therefore, a bad material for fire-resisting work. Where employed, it should have at least 9 inches of brickwork built about it, or be encased with terra-cotta blocks, a method often employed for columns.

Wrought Iron and Steel.—These two materials when heated have a tendency to twist, due to the softening of the material and the consequent reduction of the resistance to tension and compression, which is aggravated if the girders or members are fixed; it is important that all such ironwork should be allowed room for expansion, otherwise the walls will be subjected to an overturning thrust, or the iron or steel construction should be thoroughly and solidly encased with some incombustible and non-conductive material, such as concrete, brick, or terra-cotta, to prevent the metal work getting overheated.

Timber and Slag Wool Slabs.—Timber floors and half-timber houses are rendered much more fire-resisting if the intervening spaces are lined or filled in with slag wool slabs.

Concrete is a compound of broken stone, brick, ballast or other hard material and some cementing material to bind the mass together. The cement is known as the matrix and the other constituents the aggregates.

Matrix.—The cementing material employed is now invariably Portland cement. There is the ordinary Portland, the Rapid Hardening and the Aluminous cements. The latter two are employed where speed is the important factor in the work under consideration. These cements have been discussed in a previous chapter. Cement to comply with the British Standard Specification should be specified for all concrete work.

Aggregates.—Various materials are employed for the mass of the concrete, the selection depending upon the purpose for which the concrete is to be used. For concretes where strength and durability are the main factors, ballast, broken brick and broken sandstone are the materials most suitable. For many other purposes coke breeze, clinker, burnt ballast, and pumice are employed, but any material containing sulphur should be rigidly excluded for any concrete work.

Bricks.—Broken bricks form an excellent material for an aggregate providing that care has been taken in the selection. Old bricks for concrete should be of the dense, hard, vitrified type, free from any trace of sulphur or lime from the pointing material or plaster; the material should be screened free from all dust from the crushing process. Porous bricks may be employed where strength is not the primary characteristic required, but such bricks should always be well wetted before mixing, to avoid the absorption of the water required for the setting of the cement.

Broken Stone.—This makes an excellent aggregate if the type is carefully selected for the purpose for which it is required. The sandstones are generally the strongest and should be employed where the highest measure of fire-resistance is required. „Limestones in case of fire are liable to become calcined. The broken particles should be carefully graded and freed from all dust.

Furnace Slag makes a good aggregate for fire-resisting purposes, it makes a strong, heavy concrete; but should be used with care owing to its liability to contain sulphur. It is most suitable for the manufacture of partition slabs or concrete blocks.

Coke Breeze is an inexpensive aggregate for slabs and blocks, it is light in weight, but is liable to contain sulphur, and is very porous. As nails can be driven into it, it is frequently used for fixing bricks, for joinery work. Furnace slag and coke breeze should never be employed for reinforced work.

Ballast.—River or pit ballast is probably the best material for constructional work. It has a high crushing

value and is non-absorbent, and when properly graded and mixed with the right amount of sand and cement makes a very dense, and non-absorbent concrete, with a high crushing strength, and is hard, durable and waterproof. Ballast from the seashore should not be employed for any building work above ground.

Sand.—Sand is one of the products of the disintegration of rocks. In general there are two compounds resulting from such disintegration, sand and clay. Sand consists of grains of siliceous origin and occurs in beds in various conditions of purity. The most usual foreign matter being clay. The sand for building purposes should be clean and sharp. As to the first condition: sand is rarely found quite clean, a small percentage of loam is not harmful and may increase the water-resisting property of the mortar. The Reinforced Concrete Committee of the Building Research Board consider that a sand is satisfactory if it contains up to 6 per cent. of silt. The working test recommended is to fill a graduated tube with 100 c.c. of sand and 50 c.c. of water, shake thoroughly and then let the contents of the flask settle for an hour, by which time the sand will have settled at the bottom, the silt above and the water on top. The percentage of silt can then be computed from the thickness of the silt band.

The sizes of the sand grains vary. It is now customary to wash and screen the sand and ballast when excavating it. All that material passing through a $\frac{3}{16}$ inch mesh is considered sand. For the coarse aggregate for reinforced work the material should be retained on a $\frac{3}{16}$ inch mesh and pass through a $\frac{3}{4}$ inch mesh. If the space between the reinforcing bars is 1 inch then a smaller aggregate than $\frac{3}{4}$ inch should be employed. For mass concrete a larger aggregate such as would pass through a 2 inch mesh can be used.

Density.—It is imperative that reinforced concrete and concrete that is to be waterproof should be as dense as possible. To realize this property it is necessary to determine the percentage of the voids in both the sand and the coarse aggregate. The ordinary field method for determining voids is to fill a box of known capacity with the

sand or coarse aggregate, well shake it to consolidate, and then pour in water till it reaches the top of the material. The measure of the water will be the measure of the voids. The box should be well shaken to expel all air bubbles. It will generally be found that with sand and ballast graded as stated previously, the voids in both the sand and ballast will be approximately 50 per cent. So that to obtain a perfectly dense mortar the cement and sand should be in the proportion of 1 to 2. This should ensure that there is a film of cement round every particle of sand. In a like manner the proportion of the mortar to the coarse aggregate should be 1 to 2 so that the standard proportion of the three materials should be 1–2–4. This represents the leanest mix for reinforced work.

It is usual to proportion the sand and ballast by volume. Cement, being very finely ground, contains a considerable quantity of air about the particles and there is a large difference in the weight of a given quantity by volume according to whether it is lightly shovelled into the measure or is well shaken down, so that it is usual now to give the quantity by weight. One cubic foot of cement is taken to weigh 90 lbs.

Water.—Water plays an important part in the setting and hardening of concrete and is dealt with in the chapter on Limes and Cements. Clean fresh water only should be employed, free from all deleterious materials. Too much water will give a weak and porous concrete, but sufficient must be supplied to bring about the necessary chemical reactions for the setting action.

Slump Test.—This is a test to determine the best proportion of water to add to obtain the maximum strength. For this a conical-shaped sheet metal mould is required 8 inches diameter at the bottom, 4 inches diameter at the top and 12 inches in height. This is filled with the mixture. Four inches in depth at a time is punned with a $\frac{5}{8}$ inch diameter pointed rod 30 times, another 4 inches is added and is again punned, then a final 4 inches. The mould is then raised and the mass settles, the difference in height between the original height of the mass and the slumped height is

measured and is known as the slump. It is found that the driest mix such as would give a slump of about 1 inch is the proportion of water that would give the strongest concrete, but this is altogether too dry for any practical purpose, more water must be added, the amount varying with the purpose for which the concrete is required. For reinforced work where it is necessary that the mix should flow easily about the reinforcements, sufficient water to give a slump of 6 inches is required; this would require about 25 per cent. of water, but for mass concrete less water would be needed. Tests should be made for the minimum quantity of water required to give the maximum efficiency in working and the maximum strength.

For testing and for the permissible stresses for concretes of various mixes see chapter on Ferro-concretes.

Mixing.—The whole object in mixing is to produce a thoroughly homogeneous mixture, the various ingredients thoroughly distributed throughout the mass. There are two methods of mixing—hand and machine. The hand mixing is resorted to where the quantity required is not large; in this process a platform is prepared usually of scaffold boards. About 4 or 5 sleepers are first bedded on the ground and levelled. The scaffold boards are then placed in position and fixed by driving pegs on the outside edges into the ground or in any other way to prevent any displacement. It is important to ensure that the various batches are uniform mixtures, that the constituents should be accurately measured—for this, bottomless boxes of a known capacity should be employed. The measure for the ballast is first placed near one end of the platform. When the ballast has been filled into this measure, a second measure is placed on the top and filled with sand. When the boxes are removed the heap subsides in a conical heap on the boards. The sand should be spread uniformly over the ballast. The measures are constructed of a capacity to take a quantity of cement of a known weight. Cement is now distributed from the works in paper bags containing 1 cwt. of cement, which is a convenient amount for handling. The batch should therefore be proportioned to take either one or two bags of cement. The bags are opened and spread

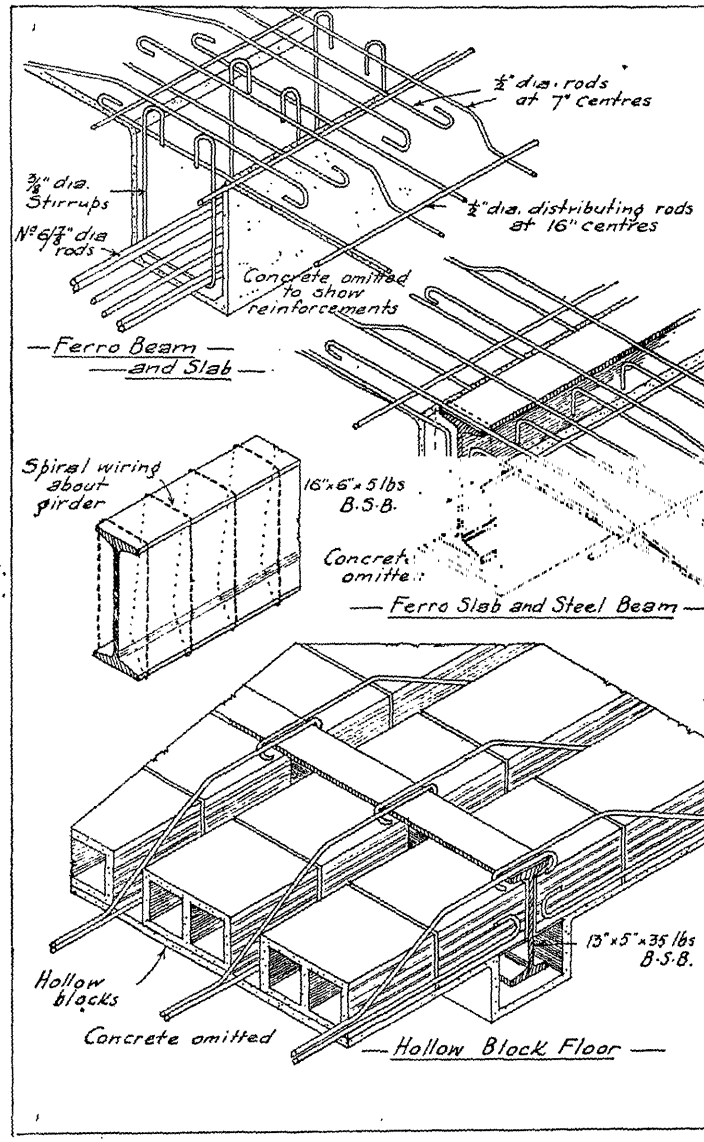
in a uniform thickness over the aggregate. The dry compound is then turned over at least twice in a dry condition, before the addition of the water. The water should be added to the required amount through a rose-headed watering can, and the material turned over twice in a wet condition. The concrete should by this time be in a fairly homogeneous condition: it should then be barrowed to the place of deposition, and worked or tamped into its position, either with the shovel or tamping rods.

Machine Mixing.—There are a large number of mechanical mixers on the market, the essential member in each consists of a revolving drum, fitted internally with projecting fins. Into these the measured quantities of the coarse and fine aggregates and water are admitted. The drum is made to revolve a given number of times, so that the materials become thoroughly mixed, and what is most important is that each batch is perfectly uniform in character. At the given time the material is discharged into barrows or chutes and deposited into position. The machine mixing gives far better results in every way, it reduces cost and wherever the magnitude of the operations justify the use of machines they should be employed.

Floors are of four types: (*a*), those consisting of main girders and filler joints, small rolled-steel beams placed at 2 ft. 6 in. centres, the spaces between being filled with concrete. All the steelwork is required by the Code to be surrounded with concrete of a 1–2–4 mix, 1 inch in thickness on the top; in the case of the main beams the sides and bottom must be provided with a 2 inch thickness of concrete and the filler joints must have a 1 inch covering on the soffit, see Fig. 453.

(*b*), *Ferro or Steel and Ferro Slabs.*—The slabs between the main joists may consist of reinforced concrete. These are less costly than the filler joists and are lighter in weight. The main beams may be either of ferro-concrete or steel. See Figs. 454 and 455. Ferro slabs and beams are treated in Chapter XXI.

(*c*), *Hollow Block Floors.*—These are usually employed in conjunction with steel main girders. This type has a


½" dia. rods at 7" centres
⅜" dia. Stirrups
No 6 ⅞" dia rods
½" dia. distributing rods at 16" centres
Concrete omitted to show reinforcements
— Ferro Beam — and Slab —
Fig. 454.
Spiral wiring about girder
16" x 6" x 51 lbs B.S.B.
Concrete omitted
— Ferro Slab and Steel Beam —
Fig. 455.
Fig. 453.
Hollow blocks
Concrete omitted
13" x 5" x 35 lbs B.S.B.
— Hollow Block Floor —
Fig. 456.

Figs. 453—456.

value from the sound as well as the fire-resisting standpoint. There is also a certain economy in the centering required. The floor consists essentially of a series of shallow T-ribs, placed as in the example shown at 1 ft. 4 in. centres. The terra-cotta blocks are placed in rows, their ends butted, and spaced the thickness of the concrete rib apart. The supports for the ribs usually consist of stout planks spaced the distance of the ribs apart. The planks provide the form-work to support the bottom of the rib when the concrete is poured, and the blocks form the sides of the rib mould. See Fig. 456.

(*d*), *Precast Beams.*—These consist of hollow beams of reinforced concrete, they are made about 10 inches in width

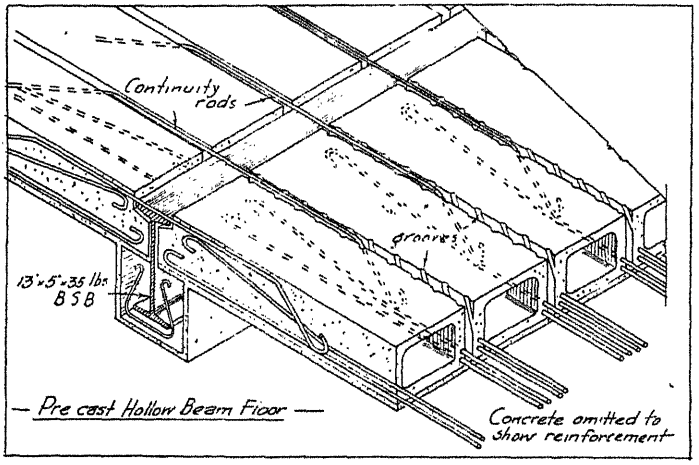


Fig. 457.

and are specially cast of the dimension and strength required for the particular case. They are placed in position between the main steel beams, their sides, which are grooved, nearly in contact. They are then grouted together. Being hollow they are sound-resisting. These beams are cured on the works before delivery. The floor can be used as soon as the grouting is set. They require no centering. See Fig. 457.

Fig. 458.

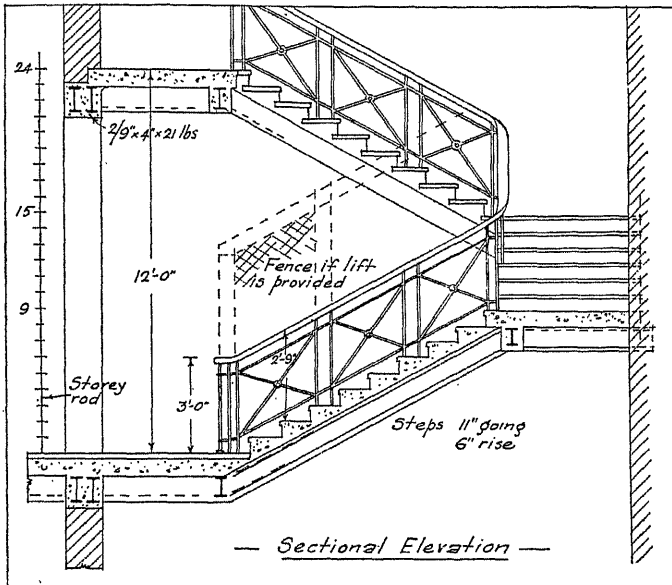
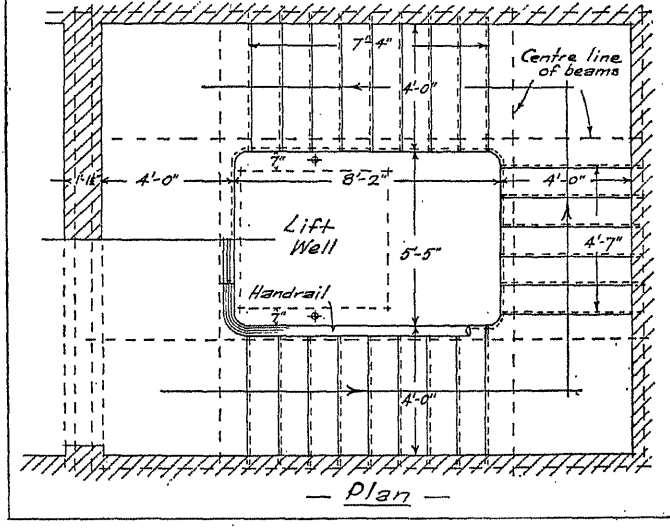


Fig. 459.



Figs. 458—459.

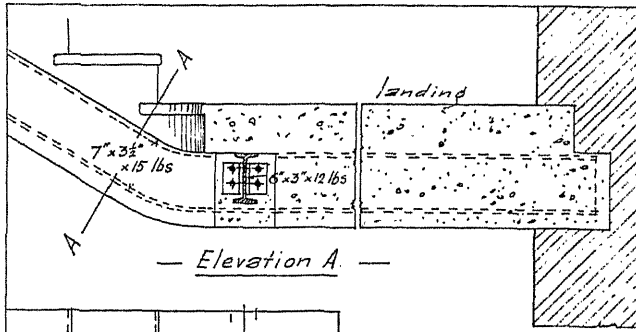


Fig. 460.

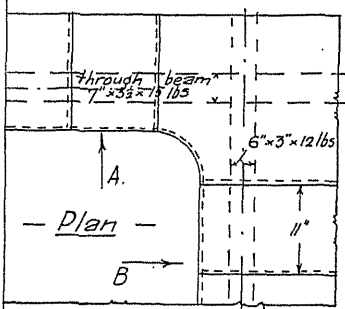


Fig. 461.

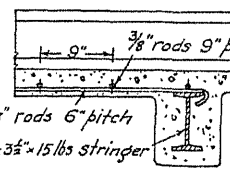


Fig. 462.

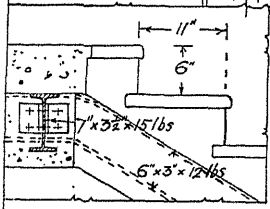


Fig. 463.

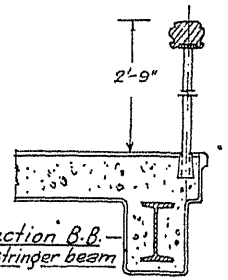


Fig. 464.

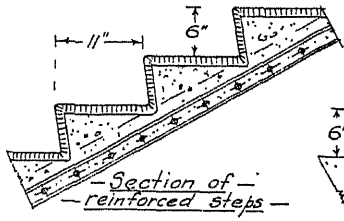


Fig. 465.

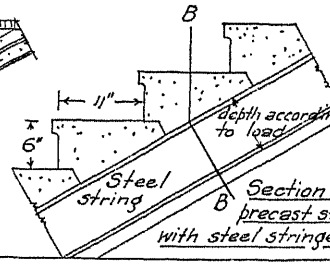


Fig. 466.

Figs. 460—466.

Stairs.—Fire-resisting stairs are now mainly of three types: (1) Concrete and steel (Figs. 458–466). (2) Ferro concrete stairs (Figs. 467–469) and pressed steel stairs (Figs. 470–477).

In the first type steel joists form the constructional members. The steps are of two kinds: (1) Precast steps, which may be built into a wall at one end and supported on a steel girder at the free end, the steel girder having first been enclosed in concrete; or (2) the steps may be cast *in situ*—wide steps of this description are usually reinforced.

Ferro Stairs.—These are cast *in situ* like the former; they may, if narrow, be of the slab type. These would usually be supported on landing beams at the top and bottom of the flight and consist essentially of a reinforced inclined slab with the steps formed above. If the steps are wide they would have a stringer beam along the outer edge of the steps or slab (see Figs. 467–469).

Messrs. Fredk. Braby & Co. Ltd. have introduced a patent pressed steel staircase (see Figs. 470–477). Although the pressings are of light gauge steel they have a very high stress resistance due to the form of the design. They also obtain an increased resistance from the enveloping material used in conjunction. Each tread and riser is formed from a single pressing, the treads being recessed to contain a filling such as granolithic, concrete, marble, slate, wood or other composition more suited than steel to give a secure foothold.

The steps are bolted together at the foot of each riser and at the back of each tread (see Figs. 474, 476, 477).

The risers thus connected are supported on both edges by stringers, again formed of light gauge Z sections, the whole being connected by tie rods, passing under each tread and connecting both stringers.

Where necessary, the wall strings are fitted with a light angle section at bottom to take the plaster finish of the walls.

Any type of ornamental balustrade can be used in conjunction with the stairs, or alternatively a solid balustrade of steel sheets as indicated on Fig. 477. This

Fig. 467.

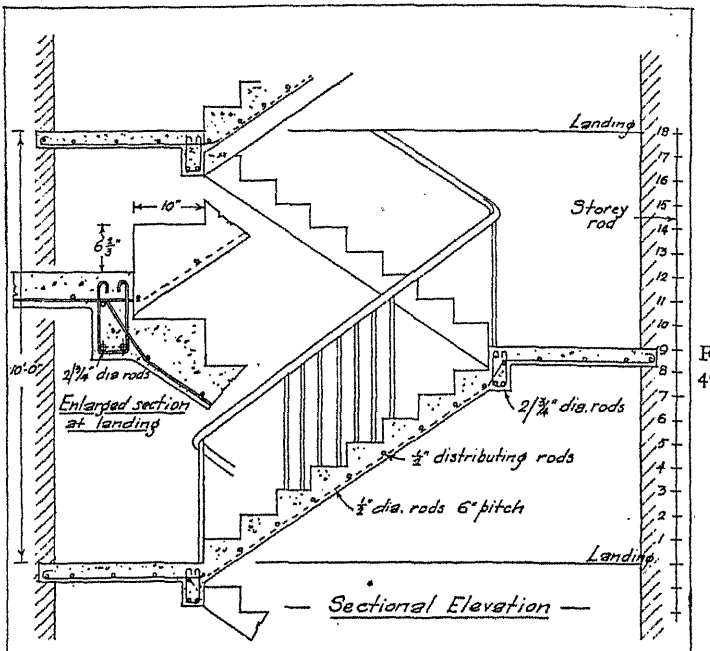


Fig. 468.

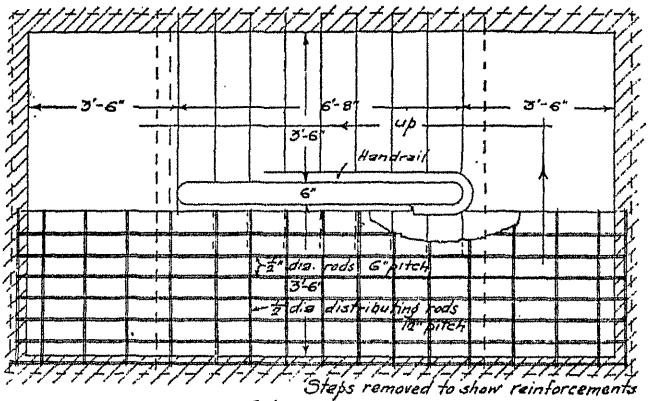
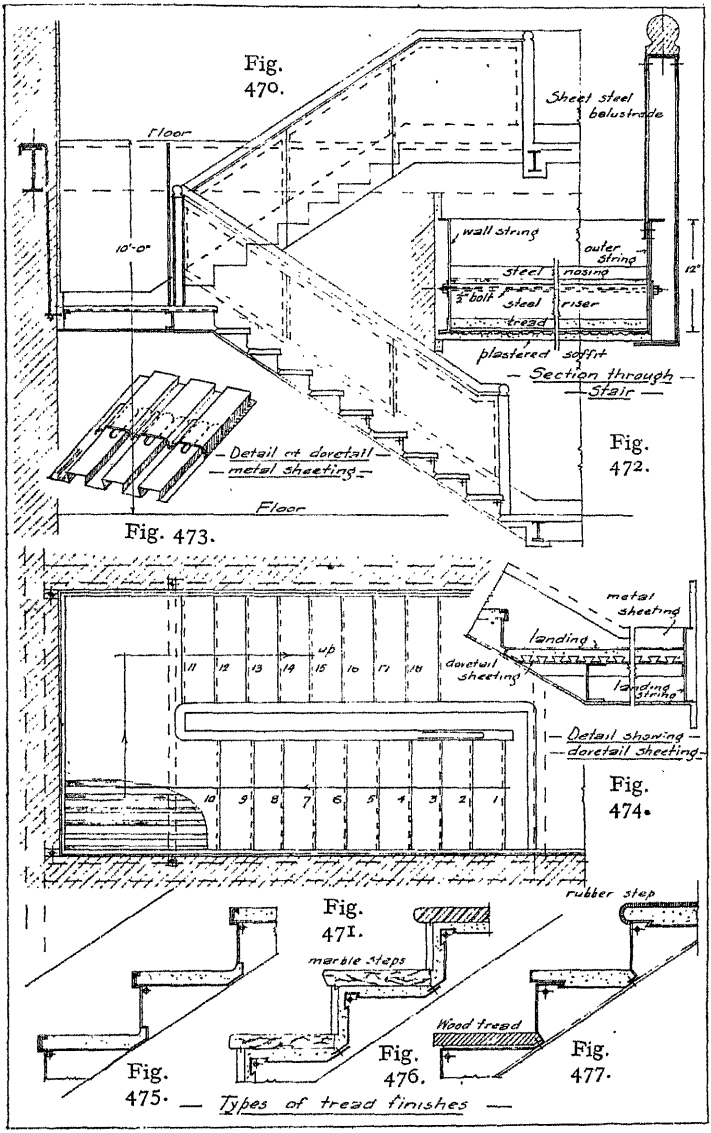


Fig. 469.

Figs. 467—469.


Fig. 470.
Sheet steel balustrade
floor
10'-0"
wall string
outer string
steel nosing
12"
steel riser
tread
plastered soffit
Section through Stair
Fig. 472.
Detail at doretail metal sheeting
Floor
Fig. 473.
metal sheeting
landing
doretail sheeting
landing string
Detail showing doretail sheeting
Fig. 474.
rubber step
Fig. 471.
marble steps
Wood tread
Fig. 475.
Fig. 476.
Fig. 477.
Types of tread finishes

Figs. 470—477.

is carried on channel uprights with a continuous channel at the head connecting the top edges of the sheets, and prepared to take a wood handrailing. Again balustrades can be formed of dovetail sheeting which can be rendered in cement, hard plaster or finished in faience or marble.

The soffits of the flights, if necessary, are fitted with a very light dovetail lathing, to which plaster can be directly floated.

The treatment of the stair depends on the type of building in which it is being used. In public buildings it is customary for all steel to be enclosed, giving the appearance of a solid concrete or marble stair.

For service stairs and general use, the steel is left open with a cement or other filling being used on the tread.

One of the great advantages of the construction is that all material is pre-fabricated in the shops before despatch and can be rapidly erected at the site in conjunction with structural steelwork before surrounding walls or concrete floors have been laid, thus giving easy access to all portions of a building at the immediate commencement of the job.

These stairs have been passed by the London County Council and have been used in many prominent buildings in the London area and in the Provinces.

CHAPTER XXI

REINFORCED CONCRETE OR FERRO-CONCRETE

FERRO-CONCRETE is the term applied to all combinations of steel and concrete in which both materials function in resisting the stresses induced in the structure. The steel resists the tensional and shearing stresses, and the concrete the compressional stresses.

The two materials are admirably suited to resist the respective stresses to which they are subjected. The Portland cement also has a preservative effect on the steel, to which it adheres with great tenacity. A point of great importance in materials to be combined for this purpose in which the adhesion of the two materials is fundamental, is that the difference in length due to expansion from changes in temperature shall be negligible. It happens that the coefficient of expansion of these two materials is almost identical, thereby nearly eliminating internal stresses from this cause.

The advantages of this method of construction are the great measure of fire-resistance where suitable materials are employed for the aggregate, and economy in construction, compared with other forms of steel and concrete combinations. Its monolithic character, in cases of soft and unreliable soils, and the unperishable nature of the materials when exposed to atmospheric conditions or in positions in which steel and many other materials are liable to corrosion and disintegration, are properties of great value.

Materials.—The subject of concrete has been discussed in a previous chapter, and it is only necessary here to emphasize the importance of the proper selection and treatment of the material, to ensure that the resulting compound

is the best of its kind. The best aggregate is shingle broken to pass a mesh of $\frac{3}{4}$ inch, granite chippings or broken sandstone. Limestone, on account of its change under the action of fire, should on no account be used. For a hard, strong concrete it is necessary that a hard, strong aggregate be employed, and that the aggregate be properly graded, varied in size so as to be retained on a mesh of $\frac{1}{8}$ inch and to pass a mesh of $\frac{3}{4}$ inch. The sand should be coarse and sharp and graded in dimension from $\frac{1}{8}$th downward. All ballast and sand should be absolutely clean and free from all earthy or clayey matter. The Portland cement should comply with the conditions of the British Standard Specification.

As concrete depends for its quality upon its density it is necessary that the constituents should be correctly proportioned, *i.e.*, that all voids in the aggregate should be completely filled with the particles of sand and cement, and in a like manner that the voids in the sand should be entirely filled with cement. The exact proportions depend upon the materials to hand, but, in general, a compound consisting of one part cement, two parts sand, and four parts of ballast will give the desired result.

Careful tests should be made to ascertain the proportions of voids to solids in the sand in order to determine the amount of cement to use, and in a like manner the ballast should be tested for voids to determine the amount of the sand and cement mixture required to give the maximum density to the compound. The homogeneity of concrete is of great importance to ensure uniform strength throughout the mass when it is set. This property is obtained by perfect mixing, which latter is only possible when a machine mixer is employed. Concrete mixed by hand is always more or less variable in quality throughout the mass.

The quantity of water employed largely affects the density and homogeneity of the mass. If an excess of water is used voids are left when evaporation takes place, and there is always a tendency for the cement and the finer portions to rise to the surface and the heavier aggregate to sink to the bottom of the mixture. The exact amount of water to add is always difficult to determine, as there is generally a certain amount in the sand. The strongest concrete for ferro work, in which the ability to flow round

the reinforcements is important, is probably obtained when from 16 to 20 per cent. of water is added to the other constituents. If the compound is too dry there is always a difficulty in tamping it properly about the reinforcements and in completely filling the moulds. A wetter mixture of between 20 to 25 per cent. is advisable to ensure that the reinforcements are completely surrounded and covered with the cementitious material. One of the most important factors in obtaining the best work is the tamping of the concrete about the reinforcements, to ensure that there are no air bubbles left in the mass. Special tools, bent to enable the mass to be worked in awkward corners, should be available. The concrete should be mixed in batches, at the same rate that it can be manipulated in the moulds. Whenever it is necessary to stop the deposition, as in the case of leaving off work for the night, the surface should be roughened and wetted and have a coat of Portland cement grout before resuming the concreting.

Reinforcements.—The reinforcements should be of mild steel and the steel should comply with the B.S.S. ; see also Code of Practice.

Several sections have been devised to grip the concrete and prevent slipping, but the ordinary circular section, owing to its greater adaptability and concentration of its mass, is most suitable for all purposes, and the high adhesive value of concrete to steel renders any indentation or other friction-forming device unnecessary. The steel bars should be free from all oil, paint, or loose scale. Rust on the surface is immaterial, though all loose rust should be removed, and the bars coated with a wash of Portland cement grouting. This forms a coating of ferrite of calcium which prevents corrosion of the steel. The important physical property of concrete and steel, which renders their combination in structural work possible, is the close approximation of their coefficients of expansion, that of steel being ·0000069 for 1° Fahr., and of concrete ·0000055. This would mean that in a length of 100 feet, and for a difference of 100 degrees, there would be a difference in length of the two materials of about ·166 inches, a quantity practically negligible.

The fundamental principle underlying ferro-concrete design is to proportion the two materials and place them, so that under any stress the stretch or deformation will be the same. This can only be done approximately, as the modulus of elasticity of concrete E_c is uncertain; it is usually taken from 2,000,000 to 2,500,000 lbs. per square inch, according to the proportion of the cement in the mixture. The value of E_s for mild steel is fairly constant and may be taken at 30,000,000 lbs. per square inch. The ratio of the concrete to the steel is termed the Modular Ratio or "m." $m = \frac{E_c}{E_s}$ = values varying from 12 to 18. E_c varies with the mix.

Testing.—Before the detailed designs for an important work are prepared, and during the execution of such a work, test cubes, 6″ × 6″ × 6″ of concrete should be made from the cement, sand and aggregate to be used in the work mixed in the proportions specified. See Appendix VII, Code.

The compression test on cubes is the Standard test. From this the other properties can be obtained, see Grading tables Code.

Loading tests on the structure itself should not be made until at least two months have elapsed since the laying of the concrete. The test load should not exceed one and a half times the accidental load. Consideration must also be given to the action of the adjoining parts of the structure in cases of partial loading. In no case should any test load be allowed which would cause the stress in any part of the reinforcement to exceed two-thirds of that at which the steel reaches its elastic limit.

Ferro buildings of ordinary type are usually constructed of a skeleton framework of columns and beams, the spaces between the latter being filled in with reinforced slabs to form the floor surfaces. If the spaces between the columns are sufficiently large, secondary beams are inserted between the main beams to reduce the span of the floor slabs. The walls may be formed of ferro, or the spaces between the columns may be filled in and the columns enclosed by brick or stonework, the whole construction in this respect being similar to steel frame construction. The monolithic charac-

ter of a ferro framework, while it gives great rigidity to a structure, renders the accurate analysis of the stresses practically impossible and many assumptions have to be made to render the calculations practicable, as for instance —the full advantage cannot wisely be taken of the fixity of the beam ends, nor can the design of the reinforcements be made to suit every variation of loading, and what may or may not be done in this respect can only come with experience in the manipulation of this material, but the examples given later will form a good general guide to the principles employed in ferro design.

Piles.—Ferro-concrete is eminently suitable for piles and any position where any alternation of wetness and dryness, which is so destructive to timber, occurs, also where erections would be subject to the ravages of insects. Both guide and sheet piles are now largely used. These piles lend themselves admirably to be lengthened during the process of driving should this be found necessary. A description of piles has been given in the chapter on Foundations.

Foundations.—The construction of reinforced foundations for soft soils is on similar principles to those described for floors.

For light buildings, the 9-inch slab of concrete often specified may be greatly strengthened by the introduction of lattice work of light steel bars from 1 to 3 feet apart or of expanded metal, placed about the centre of the thickness.

Walls.—These are constructed on the pier and panel system, the piers acting as pillars to support the various floors and roofs of the buildings. The panels form the screens between the piers. The reinforcements in the screens are usually placed in the centre.

Floors.—These are constructed, as shown in Fig. 502, similar to the triple joisted system of wood flooring, having primary girders, secondary girders and floor slabs.

Roofs and Arches.—These, if flat or having an inclination, are constructed on similar principles to floors.

Arched Vaults.—These usually have main arches constructed at intervals, the intervening vaulted surface being reinforced with interlacing bars of small diameter, or if the vaulted space be of small span, expanded metal may be used as the reinforcement.

Domes.—These may be constructed on the principle of a shell of uniform thickness, or secondly, as an arched rib vault, stiffened with horizontal rings or binders, and filled in with slabs.

The first method may be graphically calculated by treating the dome as a semi sphere, the radius being from centre of sphere to centre of thickness, then divide the height into a number of equal parts, and the sections of hollow sphere made by planes parallel to diametral plane, and containing the equal divisions of the height, will be of equal surface area. Therefore the dead loads on each ring will be equal, and the wind stress will only vary in the ratio of the normal components of wind pressure to the particular inclinations.

Generally.—No welds must be made in any of the reinforcements and all bending should be done cold by the gradual application of force. Care must be taken that all reinforcements and stirrups are laid and secured in the correct positions shown on the working drawings; these should be carefully inspected by a responsible person. Concrete must be deposited directly after mixing, and is on no account to be used after it has commenced to set. Wherever it is necessary to interrupt the work of beams, floors, etc., before completion, the surfaces must be roughened with a cutting tool and thoroughly cleansed from foreign matter; cement grout must then be poured on the surface before the work is resumed, and care taken to ram the fresh concrete as hard as possible to the old work. The proper place for stopping ferro-concrete work should always be decided by a competent person. Fresh concrete work should be freely watered or kept moist for several days, especially if exposed to heat.

Following is the Report of the Reinforced Concrete Committee with their recommendations for a Code of

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REPORT OF THE REINFORCED CONCRETE STRUCTURES COMMITTEE *

To the Building Research Board

1. We, your Reinforced Concrete Structures Committee, were appointed in September, 1931, with the following terms of reference :—

> To review present methods and regulations for the use of reinforced concrete in building and to make recommendations for rules of practice embodying the best available technical information and experience.

2. Originally, our membership consisted of the Chairman and one member of the Building Research Board and two nominees from each of the following bodies :—the Institution of Civil Engineers, the Royal Institute of British Architects, the Chartered Surveyors' Institution, the Institution of Structural Engineers, the Institution of Municipal and County Engineers and the Institute of Builders. Subsequently, in April, 1932, a member was appointed on the nomination of the Institution of Water Engineers ; in June, 1932, another member was appointed on the nomination of the Incorporated Association of Architects and Surveyors ; and in August, 1932, a nominee was appointed from the Reinforced Concrete Association.

3. We were informed at our first meeting, on 6th November, 1931, that, apart from the national need for authoritative rules of practice, the London County Council had specifically asked to be furnished, in connexion with the review of the London Building Act, with recommendations for a code of practice for the use of reinforced concrete in buildings similar to that drawn up by the Steel Structures Research Committee for the use of structural steel. Our attention was also drawn, later, to the legal position outside London regarding reinforced concrete for building, which is that detailed regulations are not in force, such bye-laws as exist being [illegible] ; and it was emphasized that our work was limited, by our terms of reference, to the preparation of technical rules of practice ; we were in no way concerned with administrative action thereon. At the same time, we were led to hope that our code of practice would be of service to Local Authorities in the administra-

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tion of their bye-laws, and that reference would be made to it in the model bye-laws issued from The Ministry of Health.

4. We decided to appoint a Sub-Committee to proceed with the drafting. Members were invited to send in suggestions to the Sub-Committee. At the outset, the Sub-Committee consisted of Mr. D. Anderson (Chairman), Mr. E. S. Andrews, Dr. O. Faber, Sir George Humphreys, Mr. B. L. Hurst and Mr. R. H. H. Stanger. Later, Mr. E. W. Butler, of H.M. Office of Works, Mr. W. L. Scott, of Considere Constructions, Limited, and Mr. H. E. Steinberg (with Mr. D. H. Green as his deputy), of the Reinforced Concrete Association, were co-opted. The Sub-Committee pressed on with the work as rapidly as circumstances would allow and held thirty-five meetings.

5. In response to the invitation to our members to forward special suggestions for consideration by the Sub-Committee, Mr. H. C. Ritchie submitted a memorandum on certain technical matters concerning primarily the application of reinforced concrete construction to water-retaining structures. Our Technical Officer, Dr. W. H. Glanville, also prepared a number of notes, besides analyses of the provisions of regulations at home and abroad and of the accumulated data at the Building Research Station.

6. We have thought it desirable to limit the scope of the present recommendations to a code of practice for the use of reinforced concrete in buildings (other than reservoirs, tanks and structures designed for the storage of fluids) wherein the loads and stresses are transmitted to the foundations by a framework of reinforced concrete or partly by a framework of reinforced concrete and/or party walls, bearing walls, or bearing structures. To have included all types of structures would have entailed more prolonged consideration and the production of a very elaborate document. Many of the provisions of the code will, however, doubtless apply to other forms of structures.

7. Existing codes of practice can be classified, according to the manner of presentation, into two main groups. Group I consists of those which are largely mathematical in form and which in general state very fully the assumptions on which the theory of design is based; the formulæ derived from this theory; the working stresses and other constants to be used in the various formulæ; the bending moments to be assumed. Group 2 are those which avoid formulæ as far as possible and state, in general terms, the basic theory of design and the constants and other quantities to be used in design; apart from a few simple exceptions that vary in the different codes, all other matters are left in the hands of the designers. Our preference has been towards Group 2. In our recommended code, therefore, formulæ have not been given where they constitute part of ordinary theory or where they flow directly from the assumptions stated.

8. We have considered the desirability of including as an appendix to the code some rules as to the practice to be followed

in submitting plans and calculations to Building Authorities. But we have not seen our way clear to preparing such an appendix, partly because we were unaware of the underlying reasons for the present variations in practice, partly because it would have meant attempting to adduce more detail than was elsewhere embodied in the code, but chiefly because we were anxious to complete our draft within a reasonable time.

9. Our recommendations are based on all the existing knowledge and experience we have been able to collate. From time to time there has become manifest the need for further experimental investigation before reliable conclusions could be reached. But, as the Board are aware, there are no funds available at present which allow these investigations to be undertaken. It has therefore been necessary to make some arbitrary decisions. In making our decisions we have constantly kept in mind the desirability of framing such recommendations as would prove acceptable, on the evidence supplied, to the responsible Building Authorities, whilst being ever anxious to leave open avenues to future progress.

10. We have not attempted legal phraseology but have endeavoured to make our intentions as clear as possible from the engineering point of view. As the benefits to be derived from the code of practice will depend on the interpretation and application of the principles given and on the general mode of administration, we venture to emphasize that special consideration should be given by the authorities concerned to such matters of interpretation and administration. We would also urge the desirability of arrangements being made to ensure that the code will be revised, at appropriate intervals, in the light of advancing experience.

11. In submitting the code, we have thought that the Board would find it useful to have before them an explanatory statement of the principles that have governed our findings. This statement may perhaps be suitable for publication with the code.

12. In conclusion, we desire to record our thanks to our Secretary, Mr. A. Zaiman, and to our Technical Officer, Dr. W. H. Glanville, of the Building Research Station. Mr. Zaiman's assistance in obtaining information from various authorities at home and abroad, in preparing numerous documents, in keeping our records and in guiding us on relations with other bodies, has been greatly appreciated and has enabled the work to proceed very smoothly. In addition to furnishing us with special notes, as already mentioned, Dr. Glanville has prepared the many drafts of various clauses as required, and has been unremitting in his efforts to ascertain by personal consultations the individual views of the members and to establish a basis for agreement. His expert assistance has been invaluable to us.

GEORGE HUMPHREYS,

(Chairman, Reinforced Concrete Structures Committee).

July, 1933.

RECOMMENDATIONS FOR A CODE OF PRACTICE FOR THE USE OF REINFORCED CONCRETE IN BUILDINGS

EXPLANATORY STATEMENT

The Reinforced Concrete Structures Committee of the Building Research Board, in drafting their Recommendations for a Code of Practice for the Use of Reinforced Concrete in Buildings, have not followed the general style and form of existing rules of practice. It was felt at the outset that this action was necessary in view of the great advances in knowledge of materials, methods of construction, and methods of calculation that have occurred in recent years. The principal differences of a general nature are as follows :—

(1) The order in which the various items are treated.

(2) The omission of all formulæ except those which are essential to the application of the Code.

(3) The addition of a number of Appendices dealing with standard methods of test. The importance of standardisation of these matters has long been realised in this country, but up to the present little action has been taken. These Appendices are essential to the correct fulfilment of the requirements of the Code.

(4) Permission to adopt certain methods of construction and to use certain materials is left to the discretion of the "Designer," while other practical matters are left to the "Responsible Supervisor." These terms are used merely as defined; they have no specific professional or legal designation:

(5) The Code has been framed in such a manner as to assure good practice while not handicapping progress, and to leave the door open to the adoption of new methods, etc., where these are adequately and authoritatively established by tests or otherwise.

Other major departures are as follows :—

(*a*) *Grades of Concrete and Permissible Stresses.*—The Committee have given special consideration to the stresses that should be permitted for concrete. They have been impressed with the great improvements in cements and aggregates and have come to the definite conclusion that industry has at this time reached a stage in development when advantage should be given to the engineer who is prepared to spend time and money in producing consistently controlled concrete. In other words the day has passed when one stress only should be permitted for a mix regardless of the care exercised and the general level of strength achieved. With this in mind, the Committee have proposed three grades of concrete.

The first grade, [illegible] Grade, is intended for use in cases where rigid control is not exercised, and where considerations of economy or convenience are such that it becomes impracticable

to insist on tests. The stresses are regarded as conservative and representative of present practice under such conditions.

The second grade, designated High Grade, calls for preliminary tests, for greater control during the progress of the work, and for a programme of tests on the deposited concrete. The stresses for this class of concrete represent present practice in certain works in this country and the Committee are satisfied that no difficulty is likely to arise in producing the strengths demanded. The upper limit of the stresses approximates to that allowed by the Ministry of Transport for use in bridges.

The third grade, designated Special Grade, calls for an even greater exercise of control and of testing. For this grade the procedure, although common abroad, is new to practice in this country, in that the actual pre-determined strength of the concrete forms the basis for the permissible stresses. While the ratios between the works test requirements and the permissible stresses have been maintained at the same values as for the other grades, a wider margin has been allowed between the pre-determined strength and the works strength in order to allow for any still uncontrolled factors. In the case of columns where bending is important, the increase in permissible stresses is, however, more apparent than real, since the methods of calculation involve allowance for the bending, whereas in the past such allowances were not normally required.

(*b*) *Steel and Permissible Stresses.*—The Committee have considered that stresses in steel should be limited by two factors—

(*a*) the factor of safety on the yield point of the steel,

(*b*) the extent of the cracking that is likely to occur on the tension side of a member in bending.

In the opinion of the Committee an adequate factor of safety is assured by making the permissible stress approximately 0·45 of the yield-point stress. It is considered that ordinary mild steel to British Standard Specification No. 15 may safely be stressed to 18,000 lb. per sq. in. The extent of the cracking should in general not go beyond that experienced with a steel stress of 20,000 lb. per sq. in., and it has, therefore, been deemed consistent to specify a yield-point requirement of 44,000 lb. per sq. in. to provide the requisite factor of safety. Cracking due to bending is not so noticeable with low percentages of steel, since the concrete contributes a greater proportionate tensile resistance than with high percentages of steel. This is more marked in solid slabs than in T-beams or other members of reduced concrete section in the tension zone. On these considerations the tensile stress in hard-drawn steel wire and other approved high yield-point steels has been increased to a limit of 0·45 of the yield-point stress, but not more than 25,000 lb. per sq. in. when the percentage of tensile steel is less than one.

Some consideration has been given to the question of including separate clauses for deformed bars, but it has been decided that these are best covered by the general clauses for special materials and methods of construction.

(*c*) *General Assumptions for Design.*—In adopting for the design of sections subject to bending general assumptions in the form used at present, the Committee have given close attention to the effects of both shrinkage and creep of concrete under load, and have concluded that the theory as at present employed, although not presenting a true picture of the stress condition in reinforced-concrete members, will provide an adequate factor of safety. The modular ratios recommended are somewhat higher than the true values, and have been selected so that, when used in conjunction with the tabulated stresses, they will produce an adequate factor of safety. At the same time, the fact that the modular ratios are higher than the actual values is a recognition of the ability of concrete to adjust itself to stress conditions by means of creep. It has not been thought necessary to include any requirement for stress calculation after large amounts of creep have taken place in sections subject to bending, since creep results in a decrease in concrete stress and in only comparatively small increases in tensile stresses in the steel, the ultimate load-carrying capacity of a member remaining practically unaffected.

(*d*) *Bending Moments.*—The Committee have felt that the question as to what bending moments exist in actual structures is one of such complexity, and there are so many factors which might affect the result, that elaborate calculations need not be insisted upon. They have recommended, therefore, that two conditions of loading only shall be considered. These conditions approximate sufficiently closely to the worst conditions for all practical purposes.

When these conditions are allowed for in design it is frequently found that the congestion of steel at column junctions is such that it is almost impossible to place the concrete satisfactorily. Moreover, for a number of reasons it is improbable that the maximum bending moments in beams near the column will be quite so high as those calculated. It has been felt, therefore, that in such a case a more satisfactory structure may be produced by the omission of a certain amount of the steel at such points, provided that the mid-span sections are correspondingly strengthened. For this reason a percentage reduction has been permitted in the support moment on condition that a corresponding increase is made in the spans adjoining. By this means the strength of the structure will at least be maintained.

The question of bending at junctions of beams with external columns is dealt with under columns in section (*g*) of this note.

Values for the bending moments in slabs spanning in two directions at right angles have been derived by considering them as perfectly elastic thin plates. This is in accordance with the best modern practice.

(*e*) *Bond and Anchorage.*—Attention is drawn to the fact that the Committee have recommended that a hook or other definite form of bend or curve should not be demanded, but that an additional length of bar may be used instead.

(*f*) *Axially Loaded Columns.*—The formulæ for columns are radically different from those now in use. Recent investigations of the effects of shrinkage and creep of concrete have shown that the former conception of the steel acting always at a stress determined by the modular ratio is erroneous. Actually the steel stress will increase continuously during the life of the column and may even reach the yield point. Nevertheless, the strength of the column as a structural unit is unaffected, and the failing load is determined by the yield point of the steel and the ultimate load-carrying capacity of the concrete. For this reason a definite value is assigned to the steel stress, the value depending on the yield point of the steel and not on the modular ratio.

The formulæ for spirally (helically) reinforced columns are based on the results of experimental work, which have shown, first, that such reinforcement increases the ultimate strength of a column by an amount depending on the quantity and yield point of this reinforcement, and secondly, that a properly proportioned spiral is approximately twice as effective as an equal quantity of longitudinal reinforcement. In the case of columns in which the spiral contributes more than the strength of the cover outside the spiral, the contribution of the concrete to ultimate strength depends on the core area; the core area is therefore taken into account in the calculations for such columns. Furthermore, tests have shown that the spiral reinforcement is called into play to resist lateral movements. The movements are small at low loads, so that the spiral is not stressed appreciably at these loads. The lateral movements increase, however, at higher loads, and therefore, the proportion of the column load borne by the spiral also increases. The increasing lateral movements are accompanied by increasing longitudinal deformation: at high loads this deformation becomes large and spalling of the cover is found to occur. In order to ensure that neither of these conditions shall be experienced in ordinary buildings, an upper limit is placed on the contribution of the spiral to the permissible load on the column in terms of the gross cross-sectional area of the concrete and its cube strength.

No increase in column load has been allowed where lateral reinforcement in the form of ties is used, since it has been impossible to find any satisfactory evidence that such an increase would be justified.

Information on the buckling of columns is very scanty, but the recommended values are in accordance with modern practice.

(*g*) *Bending in Columns.*—In proposing rather higher working stresses than those at present in use the Committee have also had in mind the possible effects of continuity between members, and that this continuity may become more effective as stronger concretes are used. For these reasons estimation of bending effects in external columns is required and approximate formulæ are given. Simple methods of estimating the stiffness of members are permitted, and the amount of computation involved is small. In the case of internal columns supporting a symmetrical arrangement of beams of approxi-

... has been found that the difference between ... bending and direct compression is adequate in itself.

(*h*) *Flat Slabs.*—The c... h the design of flat slabs have been based on the ... recently issued by the Institution of Structural ... modifications introduced have mainly been such as ...ment into line with the Code as a whole.

(*i*) *Special Forms of Construction and Special Materials.*—Within recent years new forms of construction and new types of materials have been developed on the continent and elsewhere for use in reinforced concrete work. Many of these promise to be practicable and economical, and the Committee do not wish to exclude such progressive methods and new materials or others that may be developed in the future, when adequate proof of suitability can be produced.

(*j*) *General Building Clauses.*—The general building clauses given in Appendix I to the Code follow very closely those included in the code of practice for the use of structural steel in building recommended by the Steel Structures Research Committee and adopted by the London County Council. The main modification is in the provisions for loads; slab loads have been added, and slight alterations made in the alternative superimposed loads for beams.

(*k*) *Composite Structures and Elements of Structures.*—While the Committee feel that structural steel and concrete in composite members may provide a useful and effective method of construction, they have not been able, in the time and with the information at their disposal, to deal with such forms of structural elements. The omission is not serious in view of the clause permitting special forms of construction.

CODE OF PRACTICE FOR THE USE OF REINFORCED CONCRETE IN BUILDINGS

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SECTION 1.—GENERAL

Scope.

101.—This Code of Practice relates to the use of reinforced concrete in buildings (other than reservoirs, tanks and structures designed for the storage of fluids) wherein the loads and stresses are transmitted to the foundations by a framework of reinforced concrete or partly by a framework of reinforced concrete and/or by party walls, bearing walls, or bearing structures.

The stipulations of this code indicate the minimum requirements for buildings of normal type, and the Designer must accept the full responsibility of ensuring that the provisions of the code cover the requirements of the building owner for permanent stability and safety.

The word " shall " when used in any clause hereinafter indicates the obligation of the Designer to comply with the requirements specified ; the word " may " indicates his right to exercise his own discretion.

Where the term " Designer " is used in any clause, it implies the person responsible for the structural design of the building or portion of a building.

Where the term " Responsible Supervisor " is used in any clause it implies the person responsible for supervising the construction on behalf of the building owner.

General Building Clauses.

102.—The general building clauses which shall be complied with are given in Appendix (I) of this Code of Practice.

Definitions.

Anchorage. 103.—That portion of a reinforcing bar, and/or any attachment thereto, designed to resist pulling out or slipping of the bar when subjected to stress.

Arm of Resistance Moment. The arm of the couple formed by the longitudinal internal forces crossing a section.

Beam. A member primarily carrying transverse loads by bending, and not of considerable width relative to its thickness.

Column. A member primarily carrying axial loads by means of compression stresses.

Column Head. An enlargement of a top of a column supporting a flat slab and designed to act monolithically with the column and the flat slab.

Column Length. The distance between the upper surfaces of two floors affording lateral support to the column, or the clear distance between supports plus the lateral dimension of the column.

Column Strip. A portion of a flat slab panel of total width usually equal to one-half the panel width occupying the two areas outside the middle strip as defined below. (See Fig. 479.)

Dead Load of a Building. The actual weight of all permanent construction comprised in the building.

Diagonal Band. A band of reinforcing bars running parallel to the diagonals of a panel, in a flat slab reinforced in four directions, namely, in two directions parallel to the sides of the panel and in two directions parallel to the diagonals of the panel.

Direct Band. A band of reinforcing bars running parallel to a line along the panel edge joining the column centres in a flat slab reinforced in four directions.

Drop. A portion of a flat slab, immediately surrounding the column head, which is of greater thickness than the remainder of the flat slab panel.

Effective Area of Reinforcement in a Diagonal Band of a Flat Slab. The value obtained by multiplying the normal cross-sectional area of the reinforcement by the cosine of the angle which the band makes with the direction for which its effectiveness is required.

Effective Column Length. The length upon which the ratio of column length to least lateral dimension is calculated in Clause 702 (*a*) (ii).

Effective Depth of Beams or Slabs. The distance from the compressed edge of the constructional concrete to the centre of gravity of the tensile reinforcement.

Flat Slab. A slab with reinforcement in two or four directions supported generally by columns without the medium of beams.

Formwork. All temporary moulds and supports to the concrete during the process of setting and hardening.

Long Column. A column of such length that a reduction in stresses for buckling is required under Clause 702 (*a*) (ii).

Middle Strip. A portion of a flat slab panel of width usually equal to one-half the panel width, symmetrical with regard to the centre line of the panel, and extending throughout the entire length of the panel in the direction for which moments are being considered. (See Fig. 479.)

Modular Ratio. The ratio of the modulus of elasticity of the steel to the modulus of elasticity of the concrete.

Negative Moment. A moment causing tension at the top of the slab or beam.

Positive Moment. A moment causing compression at the top of the slab or beam.

Reinforced Concrete. Concrete in which metal is embedded in such a way that the two materials act together in resisting forces.

Short Column. A column of such length that no reduction in stresses for buckling is required under Clause 702 (*a*) (ii).

Slab. A member primarily carrying transverse loads by bending, and of considerable width relative to its thickness.

Stiffness of a Member. The value obtained by dividing the moment of inertia of a member by its length.

Structural Members or Structural Framework. Any beams, slabs or columns or assemblage of beams, slabs and/or columns provided for the purpose of supporting any portion of the load of the building or of resisting any forces imposed upon it.

Superimposed Load. All load other than dead load.

Works Tests. Tests made on concrete sampled during the progress of the work from the concrete as placed.

Notation.

104.—

A = area of longitudinal steel in column.

A_b = equivalent area of binding reinforcement in the form of a spiral (volume of spiral per unit length of column).

A_c = area of concrete in column.

A_k = area of concrete in column core measured to inside of spiral.

A_w = area of shear or web reinforcement.

D = diameter of the column head supporting a flat slab.

E_c = modulus of elasticity of concrete.

E_s = modulus of elasticity of steel.

K_b = stiffness of beam.

K_l = stiffness of upper column.

K_u = stiffness of upper column.

L = length of a flat slab panel measured from centre line to centre line of the supporting columns.

M_e = bending moment at the end of a beam framing into an external column assuming both ends encastered or fixed.

M_x and M_y = bending moments at mid span in a slab spanning in two directions at right angles.
P = total axial load on short column.
S = total shear at any section.
W = total load (*i.e.*, sum of dead and superimposed loads) on beam or slab.
a = arm of resistance moment.
b = width of rectangular beam or width of rib of T-beam.
c = concrete stress.
l = effective span of beam or slab.
o = sum of the perimeters of the bars in the tensile reinforcement of a member.
x and l_y = effective spans of a slab spanning in two directions at right angles.
m = modular ratio.
p = pitch or spacing of stirrups or bent-up bars in the direction of the longitudinal axis of a beam.
s = shear stress at any section.
s_b = permissible bond stress between steel and concrete.
s_c = permissible shear stress on plain concrete.
t = stress in longitudinal reinforcement.
t_b = stress in spiral (helical) binding reinforcement.
t_w = stress in shear or web reinforcement.
u = cube crushing strength of concrete, required from the works tests, as given in Clause 301.
w = total load per unit area on a slab.
z_x and z_y = bending moment coefficients for slabs spanning in two directions at right angles.

SECTION 2.—MATERIALS

Cement.

201.—(*a*) *Portland Cement.*—All Portland cement shall comply with the requirements of the B.S.S. for Portland cement.

Rapid-hardening Portland cement shall comply with the requirements of the appropriate B.S.S. or, in the absence of such specification, with the requirements of the B.S.S. for Portland cement.

(*b*) *Portland-Blastfurnace Cement.*—All Portland-Blastfurnace cement shall comply with the requirements of the B.S.S. for Portland-Blastfurnace cement.

(*c*) *High Alumina Cement.*—High alumina cement shall comply with the appropriate B.S.S. or, in the absence of such specification, with the physical requirements of the B.S.S. for Portland cement.

Aggregates

202.—(*a*) *General* (*Permissible Materials*).—Aggregates shall consist of natural sands and gravels, crushed stone, or other suitable material. They shall be hard, strong and durable, and shall be clean and free from clay films and other adherent coatings.

(*b*) *Prohibited Materials and Impurities.*—Aggregates shall contain no deleterious material in sufficient quantity to reduce the strength or durability of the concrete, or to attack the steel reinforce-

ment. Under this clause prohibited materials include the following :—

(i) Coal and coal residues, including clinkers, ashes, coke, breeze, pan breeze, slag and other similar material.

(ii) Copper slag, forge breeze, dross and other similar material.

(iii) Soluble sulphates, including gypsum and other similar material.

(iv) Coarse aggregate of a porous nature if the percentage increase in weight of a representative dry sample of the material exceeds 10 per cent. after immersion in water for 24 hours, excepting as permitted under Clause (*c*) below.

(v) Fine aggregate containing organic material in sufficient quantity to show a darker colour than the standard colour when tested according to the method given in Appendix (II).

(vi) Fine or coarse aggregate containing clay lumps exceeding 1 per cent. by weight.

(vii) Fine aggregate containing material removable by decantation, according to the standard method given in Appendix (III), exceeding 3 per cent. by weight.

The use of aggregates not conforming to the foregoing requirements regarding prohibited [illegible] may be specially authorized by the [illegible] shown by approved tests and to his satisfaction that the concrete produced with the particular aggregates in the proportions and consistence to be used has properties such that [illegible] durability are of the standard required by this Code [illegible].

(*c*) *Porous Aggregate.*—Aggregates of a porous nature not complying with Clause (*b*) (iv) above shall be permitted in non-load bearing reinforced-concrete members where the risk of corrosion of the reinforcement is negligible.

(*d*) *Grading.*—(i) Fine aggregate shall be of such a size that it will pass through a mesh $\frac{3}{16}$ in. square measured in the clear. Not more than 3 per cent. by weight shall pass a No. 100 B.S. sieve (see Appendix (IV) for Standard Method of Sieving).

(ii) Coarse aggregate shall be of such a size that it will be retained on a mesh $\frac{3}{16}$ in. square measured in the clear.

The maximum size of coarse aggregate shall be $\frac{3}{4}$ in. in normal cases.

Coarse aggregate of a smaller maximum size than $\frac{3}{4}$ in. shall be used where the minimum clear lateral distance between reinforcing bars is less than 1 in., and the maximum size shall then be $\frac{1}{4}$ in. less than such distance.

Coarse aggregate of a maximum size greater than $\frac{3}{4}$ in. may be used provided that this maximum size is not greater than three-quarters of the cover or of the minimum clear lateral distance between any two reinforcing bars, whichever is less.

(iii) The grading between the limits specified above shall be such as to produce a dense concrete [illegible] proportions and con-

sistence that will work readily into position without segregation and without the use of an excessive water content.

203.—(*a*) *Purity.*—Water shall be free from deleterious materials, reasonably clean, and from a source approved by the Responsible Supervisor. Water.

204.—(*a*) *Quality.*—Steel reinforcement shall comply with the requirements of B.S.S. No. 15 for Structural Steel (Quality A) or B.S.S. No. 165 for Hard-drawn Steel Wire or other Specification approved by the competent Building Authority as provided for under Clause 302 (*b*). All bars shall be tested the full size as used. When the yield-point stress is specified, it shall be determined in accordance with Appendix (V). Steel.

(*b*) *Welding.*—(i) Welding of reinforcement at joints which will be stressed in tension is forbidden. Welding of reinforcement at joints which will be stressed in compression shall be permitted when specially authorised by the Designer, and it shall then be done by experienced welders. Welding at points where transverse bars are in contact shall be permitted.

(ii) Forge welding is prohibited.

205.—Cement and aggregates shall be stored at the work in such a manner as to prevent deterioration or contamination. Any material which has deteriorated or has been damaged shall be immediately removed from the work. Storage of Materials.

SECTION 3.—STRENGTH REQUIREMENTS AND PERMISSIBLE STRESSES

301.—(*a*) *Grades of Concrete.*—Three grades of concrete designated Ordinary Grade, High Grade and Special Grade shall be recognised. The requirements for each grade shall be as follows :— Concrete.

(*b*) *Ordinary Grade Concrete.*—(i) Strength and consistence tests shall be made when required by the Designer and shall then be carried out as specified in Appendices (VII), (VIII) and (VI) respectively.

(ii) The minimum cube strengths, the permissible stresses and the appropriate values of the modular ratios for Ordinary Grade Concrete shall be as given in Table 1.

(*c*) *High-Grade Concrete.*—(i) Preliminary tests shall be made prior to the commencement of the work and in accordance with Appendix (VII) unless satisfactory evidence of strength is produced from reliable sources.

(ii) The work shall be carried out under special supervision throughout and shall be in charge of a Foreman and a Responsible Supervisor or Clerk of Works, both competent and qualified for the execution of reinforced concrete work.

(iii) Works tests for strength and consistence shall be carried out as specified in Appendices (VIII) and (VI) respectively. At least two strength cubes shall be tested weekly and whenever any of the materials or the mix is changed. At least one consistence test

shall be made daily. The first of such strength and consistence tests shall be made immediately concreting is commenced.

(iv) The minimum crushing strengths, the permissible stresses and the appropriate values of the modular ratios for High Grade concrete shall be as given in Table 2.

(*d*) *Special Grade Concrete.*—(i) The requirements for High Grade concrete given above in paragraphs (i) and (ii) shall be complied with.

(ii) The structure or portion of a structure in which Special Grade concrete is to be used shall be calculated and designed as a continuous monolithic framework, bending in all members being taken into account.

(iii) Special provision shall be made to ensure a uniform supply of cement throughout the work.

(iv) The water content, including the moisture in the aggregates, shall be controlled in such a manner that the ratio of the water to the cement will not at any time exceed the ratio used in the preliminary tests by more than 10 per cent.

(v) The grading of the aggregates shall be carefully controlled throughout the work in order to ensure that at all times it shall conform closely to that used for the preliminary tests.

(vi) Works tests for strength and consistence shall be carried out as specified in Appendices (VIII) and (VI) respectively. At least two series of two strength cubes shall be tested weekly and in addition whenever any of the materials or the mix is changed. At least one consistence test shall be made daily. The first of such strength and consistence tests shall be made immediately concreting has commenced.

(vii) The permissible stresses shall be based on the results of the preliminary cube tests and shall not exceed the values permitted for similar mixes of High Grade concrete by more than 25 per cent. Shear and bond stresses shall also not exceed 150 lb. per sq. in.

The permissible stresses, the minimum crushing strengths required from the works tests, and the [illegible] lular ratios shall be obtained from the relations given [illegible] of Table 3.

(viii) Where it can be shown that the conditions under which the work is to be carried out are favourable with regard to temperature and humidity, and such that there is no risk of temperatures less than 50° F. during the placing of the concrete and that the concrete will be maintained in a damp condition for at least 14 days, the preliminary test requirements given in Table 3 may be reduced by 20 per cent. (*i.e.*, may be taken as equal to $4x$).

(*e*) *General.*—For all grades of concrete where the strengths specified in Tables 1, 2 or 3 are reached before the age of 28 days these earlier tests may be accepted.

Where the works cube tests at the age of 28 days show strengths which fall be[illegible] in the Tables the concrete shall not be [illegible] tests at the age of 56 days show strengt[illegible] specified for the age of 28 days plus 10 per cent.

TABLE 1.—*Ordinary Grade Concrete*

Mix Reference	Nominal Mix	Proportions. Cub. ft. of Aggregate per 112 lb. bag of cement		Minimum Cube Strength Requirements at 28 Days. (Optional Tests see Cl. 301 (*b*) (i)). Lb. per sq. in.		Modular Ratio	Permissible Concrete Stresses. Lb. per sq. in.			
		Fine	Coarse	Preliminary Tests	Works Tests		Bending	Direct	Shear	Bond
				$4{\cdot}5x$	$3x$	$\frac{40{,}000}{3x}$	x	$0{\cdot}8x$	$0{\cdot}1x$	$0{\cdot}1x+25$
I	1 : 1 : 2	1¼	2½	4,388	2,925	14	975	780	98	123
II	1 : 1·2 : 2·4	1⅛	3	4,163	2,775	14	925	740	93	118
III	1 : 1·5 : 3	1⅜	3¾	3,825	2,550	16	850	680	85	110
IV	1 : 2 : 4	2½	5	3,375	2,250	18	750	600	75	100

Where other proportions of fine to coarse aggregate are used the requirements shall be based on the ratio of the sum of the volumes of the fine and coarse aggregates, each measured separately, to the quantity of cement, and shall be obtained by proportion from the two nearest defined mixes.

The tabulated values of the modular ratios are given to the nearest whole number; nevertheless, more exact values, calculated from the formula $40{,}000/3x$, may be used.

TABLE 2.—*High Grade Concrete*

Mix Reference	Nominal Mix	Proportions. Cub. ft. of Aggregate per 112 lb. bag of cement		Minimum Cube Strength Requirements at 28 Days. Lb. per sq. in.		Modular Ratio	Permissible Concrete Stresses. Lb. per sq. in.			
		Fine	Coarse	Preliminary Tests	Works Tests		Bending	Direct	Shear	Bond
				$4{\cdot}5x$	$3x$	$\frac{40{,}000}{3x}$	x	$0{\cdot}8x$	$0{\cdot}1x$	$0{\cdot}1x+25$
I	1 : 1 : 2	1¼	2½	5,625	3,750	11	1,250	1,000	125	150
II	1 : 1·2 : 2·4	1⅛	3	5,400	3,600	11	1,200	960	120	145
III	1 : 1·5 : 3	1⅜	3¾	4,950	3,300	12	1,100	880	110	135
IV	1 : 2 : 4	2½	5	4,275	2,850	14	950	760	95	120

Where other proportions of fine to coarse aggregate are used the requirements shall be based on the ratio of the sum of the volumes of the fine and coars[illegible] measured separately, to the quantity of cement, and shall be obtained [illegible] the two nearest defined mixes.

The tabulated values of the modular ratios are given to the nearest whole number nevertheless, more exact values, calculated from the formula 40 000/3x may be used.

TABLE 3.—*Special Grade Concrete (Limiting Values)*

Mix Reference	Nominal Mix	Proportions. Cub. ft. of Aggregate per 112 lb. bag of cement		Minimum Cube Strength Requirements at 28 Days. Lb. per sq. in.		Modular Ratio	Permissible Concrete Stresses. Lb. per sq. in.			
				Preliminary Tests	Works Tests		Bending	Direct	Shear	Bond
		Fine	Coarse	$5x$	$3x$	$\frac{40,000}{3x}$	x	$0{\cdot}8x$	$0{\cdot}1x$ but not greater than 150	$0{\cdot}1x+25$ but not greater than 150
I	1:1:2	1	2	Strength requirements, modular ratio and permissible stresses to be examined from above tables and to be based on preliminary tests only. The values for the permissible stresses shall not exceed those for similar mixes of High Grade concrete (Table 2) by more than 25 per cent. Shear and bond stresses also shall not exceed 150 lb. per sq. in. Modular ratios may be taken to nearest whole number.						
II	1:1·2:2·4	1	3							
III	1:1·5:3	1	3							
IV	1:2:4	2	5							

Where other proportions of fine to coarse aggregate are used the requirements shall be based on the ratio of the sum of the volumes of the fine and coarse aggregates, each measured separately, to the quantity of cement, and shall be obtained by proportion from the two nearest defined mixes.

Where works cube tests show strengths consistently above those specified the Designer may authorise a reduction in the number of tests required.

In no case should the consistency of the concrete be such as to produce a slump, when tested in accordance with Appendix (VI), of more than 6 in., and wherever possible a smaller slump should be maintained.

(*f*) *Method of Measuring Materials.*—The quantity of cement shall be determined by weight.

The quantities of fine and coarse aggregate shall be separately determined either by volume or equivalent weight.

(*g*) *Proportions.*—Concrete in the proportions given in Tables 1, 2 and 3, or in intermediate proportions, shall be recognised for all grades of concrete.

The volume of coarse aggregate shall be twice that of the fine aggregate except that when specially authorised by the Designer it may be varied within the limits of one and a half and two and a half times the volume of fine aggregate.

302.—(*a*) *General.*—The stresses in the steel reinforcement shall not exceed the values given in Table 4. Reinforcement.

Where the stresses given in column (2) are used, only steel complying with the requirements stated in the heading shall be used throughout the job.

(*b*) *High Yield Point Steel.*—In solid slabs, other than flat slabs, subject to bending, the permissible stress in steel complying with B.S.S. No. 165 for Hard Drawn Steel Wire or other Specification for high yield point steel approved by the competent Building Authority may be increased up to a value equal to 0·45 of the yield point stress but not exceeding 25,000 lb. per sq. in., providing that the area of steel in tension does not exceed 1 per cent. of the effective area of the slab.

TABLE 4

	Permissible Stress. Lb. per sq in.	
	(1) Mild Steel Complying with B.S.S. No. 15.	(2) Mild Steel Complying with B.S.S. No. 15 and with a Yield Point Stress of not less than 44,000 lb. per sq. in. See Appendix (V).
Bending		
Tension in longitudinal steel in beams, slabs or columns subject to bending	18,000	20,000
Compression in longitudinal steel in beams, slabs or columns subject to bending where the compressive resistance of the concrete is taken into account.	The compression stress in the surrounding concrete multiplied by the modular ratio.	
Compression in longitudinal steel in beams where the compressive resistance of the concrete is not taken into account	18,000	20,000
Direct Compression		
Compression in longitudinal steel in axially loaded columns	13,500	15,000
Tension in spiral reinforcement ...	13,500	15,000
Shear		
Tension in web reinforcement ...	18,000	18,000

SECTION 4.—WORKMANSHIP

Concrete.

401.—(*a*) *Mixing.*—The concrete shall be mixed in an approved mechanical batch mixer unless hand mixing is approved, when 10 per cent. extra cement shall be used.

The mixing shall continue until there is a uniform distribution of the materials and the mass is uniform in colour and consistence.

(*b*) *Transporting.*—Concrete shall be handled from the place of mixing to the place of final deposit as rapidly as practicable by methods which will prevent the segregation or loss of the ingredients. It shall be deposited as nearly as practicable in its final position to avoid re-handling or flowing.

When concrete is conveyed by chuting, the plant shall be of such size and design as to ensure practically continuous flow in the chute. The slope of the chute shall be such as to allow the concrete to flow without the use of an excessive quantity of water and without segregation of the ingredients. The delivery end of the chute shall be as close as possible to the point of deposit. When the operation is intermittent, the spout shall discharge into a hopper. The chute shall be thoroughly flushed with water before and after each working period ; the water used for this purpose shall be discharged outside the formwork.

(*c*) *Placing.*—(i) *General.*—The concrete shall be placed in its final position before setting has commenced and shall not subsequently be disturbed. A record shall be kept on the work of the time and date of placing the concrete in each portion of the structure.

(ii) *Construction Joints.*—Concreting shall be carried out continuously up to construction joints, the position and arrangement of which should be pre-determined by the Designer. Any rest pauses, such as for meals, should be subject to the approval of the Responsible Supervisor.

In the case of horizontal joints any excess water and laitance shall be removed from the surface after the concrete is deposited and before it has set.

When work has to be resumed on a surface which has hardened, such surface shall be well roughened and all laitance removed ; the surface shall then be swept clean, thoroughly wetted, and covered with a thin layer of mortar composed of equal volumes of cement and sand.

(iii) *Compacting.*—Concrete shall be thoroughly compacted during the operation of placing, and thoroughly worked around the reinforcement, around embedded fixtures, and into the corners of the formwork.

(*d*) *Curing.*—Concrete after placing shall be protected during the first stages of hardening from harmful effects of sunshine, drying winds and cold, and also from running or surface water and shocks. The concrete shall be prevented from drying out for at least 7 days.

When rapid-hardening cements are used special attention shall be given to the maintenance of moist conditions of curing ; in particular, concrete made with high alumina cement shall be kept thoroughly wet for the first 24 hours.

(*e*) *Work in Cold Weather.*—When depositing concrete at or near freezing temperatures precautions shall be taken, to the satisfaction of the Responsible Supervisor, to ensure that the concrete shall have a temperature of at least 40° F. and that the temperature of the concrete shall be maintained above 32° F. until it has thoroughly hardened. When necessary, concrete materials shall be heated before mixing. Dependence shall not be placed on salt or other chemicals for the prevention of freezing.

No frozen materials or materials containing ice shall be used.

All concrete damaged by frost shall be removed.

Steel.

402.—(*a*) *Cleaning.*—All metal for reinforcement shall be free from loose mill scale, loose rust, oil and grease, or other deleterious matter, immediately before placing the concrete.

(*b*) *Placing.*—All reinforcement shall be placed and maintained in the position shown on the drawings.

(*c*) *Bending.*—Reinforcement shall not be bent or straightened in a manner that will injure the material.

Bending hot at a cherry-red heat (not exceeding 1,550° F.) may be allowed except for bars which depend for their strength on cold working. Bars bent hot shall not be cooled by quenching.

Formwork

403.—(*a*) *General.*—The formwork shall be so constructed as to remain sufficiently rigid during the placing of the concrete.

The formwork shall be sufficiently tight to prevent loss of liquid from the concrete.

(*b*) *Strutting.*—The vertical strutting shall be maintained continuous through the lower storeys to the foundations or to other construction which is sufficiently strong to afford the required support without injury.

(*c*) *Cleaning and Treatment of Forms.*—All rubbish shall be removed from the interior of the forms before the concrete is placed, and the formwork in contact with the concrete shall be cleaned and thoroughly wetted or treated with a composition approved by the Designer. Care shall be taken that such approved composition shall be kept out of contact with the reinforcement.

(*d*) *Procedure when Striking.*—All formwork shall be removed without shock or vibration. Before the formwork is stripped the concrete surface shall be exposed where necessary in order to ascertain that the concrete has sufficiently hardened. Proper precautions shall be taken to allow for the decrease in rate of hardening that occurs with both normal and rapid-hardening Portland cements in cold weather.

Fixing Blocks.

404.—Fixing blocks may be embedded in the concrete provided that the strength or effective cover of any part of the structure is not reduced below the standard required by this Code of Practice.

Electrical Currents.

405.—No part of the reinforcement shall be used for conducting electrical currents.

SECTION 5.—DESIGN—GENERAL

General Assumptions.

501.—The calculation of the bending stresses in reinforced-concrete members shall be based on the following general assumptions :—

(i) Both steel and concrete are perfectly elastic.
(ii) Concrete carries no tensile stress.
(iii) Plane sections remain plane after bending.
(iv) The modular ratio has the values given in Clause 301.

Stresses due to shrinkage or expansion of the concrete may be neglected.

The weight of reinforced concrete may be assumed equal to 144 lb. per cu. ft.

Cover.

502.—The cover of constructional concrete measured from the outside of all reinforcing bars, including transverse ties, spirals, stirrups and all secondary reinforcement, shall at all points be at least $\frac{1}{2}$ in. or the diameter of the bar, whichever is the greater. For main reinforcing bars in beams or columns such cover shall be at least 1 in. or the diameter of the bar, whichever is the greater.

Bar Sizes—General Limits.

503.—The diameter of any reinforcing bar shall not exceed 2 in.

The diameter of any reinforcing bar, including transverse ties, spirals, stirrups and all secondary reinforcement, shall be at least $\frac{3}{16}$ in.

The diameter of main reinforcing bars in beams and slabs shall be at least $\frac{1}{4}$ in.

The diameter of longitudinal reinforcing bars in columns shall be at least $\frac{1}{2}$ in.

The diameter of wires under tensile stress in connected mesh and similar reinforcement in slabs shall be at least $\frac{1}{10}$ in.

Distance between Bars.

504.—The minimum lateral distance between reinforcing bars shall be the diameter of the bar or $\frac{1}{4}$ in. more than the maximum size of coarse aggregate, whichever is the greater, and at points of splice the bars shall be so disposed that this distance is maintained between each pair of lapped bars and adjacent bars.

The vertical distance between horizontal main reinforcing bars shall be at least $\frac{1}{2}$ in. except at splices or where transverse bars are in contact.

The pitch of bars or wires of main tensile reinforcement in beams and slabs shall not exceed 12 in. or twice the effective depth, whichever is the lesser.

The pitch of distributing bars in slabs shall not exceed four times the effective depth of the slab.

All meshed reinforcement shall be of such dimensions as will enable the coarse material in the concrete to pass easily through the meshes of such reinforcement.

The spacing for transverse reinforcement in columns and for shear reinforcement in beams shall be in accordance with Clauses 701 (*b*) and 603 (*b*) respectively.

505.—The internal radius expressed in bar diameters of a bend in a reinforcing bar shall not be less than the value obtained by dividing the stress in the steel at the commencement of the bend by four times the permissible stress in the concrete in direct compression where the minimum concrete cover is used, and not less than two-thirds this value where conditions are such that there is no danger of splitting the concrete. Bends in Bars.

506.—For the purpose of estimating the stiffness of a reinforced-concrete member the moment of inertia may be calculated on its whole section. Allowance for the reinforcement may be made using the appropriate modular ratio. In the case of a beam the breadth of the compression slab shall be taken in accordance with Clause 601 (*g*). The method employed in estimating the moments of inertia shall be the same for all members considered in any one calculation. Estimation of the Stiffness of a Member.

SECTION 6.—DESIGN—BEAMS AND SLABS

601.—(*a*) *Effective Span.*—The effective span of a beam or slab shall be taken as the distance between the main vertical sides of the supporting members plus the effective depth of the beam or slab at the supports, or the span between the centres of the necessary bearing surfaces, whichever is the lesser. General Clauses.

(*b*) *Lateral Support.*—Beams shall be secured laterally whenever the ratio of the length of the beam to the width of its compression flanges exceeds

$$20 \left\{ 3_{\frac{1}{3}} - 2 \times \frac{\text{calculated compressive stress}}{\text{permissible compressive stress under Clause 301}} \right\}.$$

(*c*) *Distribution Reinforcement in Slabs.*—For solid slabs spanning in one direction only, distributing bars shall be provided at right angles to the main tensile bars. Such distributing bars shall have an aggregate cross-sectional area of at least 20 per cent. of the main tensile reinforcement, and the pitch of such distributing bars shall be not greater than four times the effective depth of the slab.

(*d*) *Compression Reinforcement in Beams.*—Where the compressive resistance of the concrete is taken into account, the compression reinforcement shall be effectively anchored over the distance where it is required at points not further apart (centre to centre) than 12 times the diameter of the anchored bar.

Where the compressive resistance of the concrete is not taken into account, the compression reinforcement shall be effectively anchored laterally and vertically over the distance where it is required at points not further apart (centre to centre) than eight times the diameter of the anchored bar. The subsidiary reinforcement used for this purpose shall pass round or be hooked over both the compression and tension reinforcement.

(*e*) *Staggering of Bars.*—To prevent the formation of continuous cracks the points at which bars are discontinued shall be suitably staggered wherever practicable.

(*f*) *Limits of Bar Sizes.*—The limits of bar sizes shall be in accordance with Clause 503.

(*g*) *T-Beams and L-Beams.*—(i) Where in T-beams the slab takes the compression its breadth shall not be taken to exceed the least of the following :—

(*a*) One-third of the effective span of the T-beam ;

(*b*) The distance between the centres of the ribs of the T-beams ;

(*c*) The breadth of the rib plus 12 times the thickness of the slab.

(ii) Where in L-beams the slab takes the compression its breadth shall not be taken to exceed the least of the following :—

(*a*) One-sixth of the effective span of the L-beam ;

(*b*) The breadth of the rib plus one-half the clear distance between ribs.

(*c*) The breadth of the rib plus four times the thickness of the slab.

(iii) The reinforcement in that portion of slab required to take the compression in a T or L-beam shall extend its full width and shall consist of bars transverse to the beam. Such reinforcement shall not be less than 0·3 per cent. of the total cross-sectional area of the slab, and in cases where the slab is assumed to be independently spanning in the same direction as the beam such reinforcement shall be near the top surface of the slab.

Bending Moments.

602.—(*a*) *General.*—In order to estimate the bending moments on a beam or slab, the effective span and the whole load on the effective span shall be taken into account.

The bending moments to be provided for at every cross-section shall be the maximum positive and negative moments at such cross-section for the following conditions of loading :—

(i) alternate spans loaded and all other spans unloaded ;

(ii) adjacent spans loaded and all other spans unloaded.

Nevertheless, provided that the maximum positive moments so obtained in any two adjacent spans are increased by an amount not exceeding 15 per cent. of the maximum intermediate support moment, this latter may be reduced by the same amount and the bending moment curves adjusted accordingly.

The bending moments to be provided for in flat slabs shall be in accordance with the requirements of Section 8.

(*b*) *Beams and Slabs Spanning in One Direction.*—The bending moments in beams and slabs spanning in one direction shall be calculated on one of the following assumptions :—

(i) Beams may be considered as members of a continuous framework with monolithic connection between beams and columns, and the maximum bending moments calculated taking into account the resistance of the columns to bending.

(ii) Beams and slabs may be considered as continuous over supports about which they can freely rotate. Where beams

frame into external columns they shall be designed to resist a negative bending moment equal to the sum of the bending moments in the upper and lower columns, calculated in accordance with Clause 702 (*b*).

(iii) Unless more exact estimates are made, the total bending moments in cases of uniformly distributed loading over a number of approximately equal spans may be assumed to have the following values :—

Near Middle of End Span.	At Support Next to End Support.	At Middle of Interior Spans.	At Other Interior Supports.
$+\frac{Wl}{\quad}$	$-\frac{Wl}{10}$	$+\frac{Wl}{12}$	$-\frac{Wl}{12}$

where W = total load (*i.e.*, sum of dead and superimposed loads) on the beam or slab.

and l = effective span of the beam or slab.

(*c*) *Slabs Spanning in Two Directions at Right Angles.*—(i) *General.*—In order to estimate the bending moments in a solid slab spanning in two directions at right angles, the slab may be assumed to act as a perfectly elastic thin plate, Poisson's ratio being assumed equal to zero.

(ii) *Slabs Simply Supported on Four Sides.*—Where the corners of the slab are prevented from lifting and adequate provision is made by special reinforcement at the top and bottom or otherwise, to resist torsion at the corners of the slab, the bending moments at the centre of a uniformly loaded simply supported slab may be assumed to have the values given in Table 5, case (*a*), and shown by curves (*a*) in Fig. 478. Where such provision is not made, the bending moments shall be assumed to have the values given in Table 5, case (*b*), and shown by curves (*b*) in Fig. 478.

(iii) *Slabs Fixed at or Continuous Over Four Sides.*—The values given in Table 5, case (*a*), and Fig. 478, curves (*a*) may be reduced by 20 per cent. when the slab is fixed at or continuous over the four sides, provided that the negative bending moments to be provided for at the supports shall be equal to the values obtained from case (*a*) without reduction.

TABLE 5.—*Bending Moment Coefficients for Slabs Spanning in Two Directions at Right Angles*

$$M_x = z_x \frac{wl_x^2}{8}\ ; \quad M_y = z_y \frac{wl_y^2}{8},$$

where M_x and M_y = bending moments to be taken on strips of unit width, and of effective spans l_x and l respectively,

w = total load per unit area,

and z_x and z_y = coefficients.

Case (a)

l_y/l_x ...	1·0	1·1	1·2	1·3	1·4	1·5	1·75	2·0	2·5	3·0
z_x ...	0·295	0·358	0·419	0·477	0·532	0·581	0·681	0·757	0·869	0·940
z_y ...	0·295	0·237	0·191	0·154	0·127	0·107	0·071	0·051	0·032	0·022

Case (b)

l_y/l_x ...	1·0	1·1	1·2	1·3	1·4	1·5	1·75	2·0	2·5	3·0
z_x ...	0·500	0·594	0·675	0·741	0·794	0·835	0·904	0·941	0·975	0·988
z_y ...	0·500	0·406	0·325	0·259	0·206	0·165	0·096	0·059	0·032	0·022

Resistance to Shear.

603.—(*a*) *General.*—(i) The shear stress "s" at any cross-section in a reinforced concrete beam or slab shall be calculated from equation (1).

$$s = \frac{S}{ba} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (1)$$

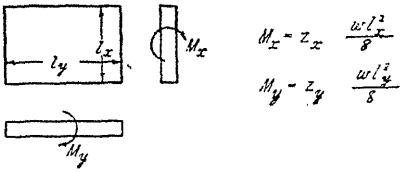
where S = total shear across any section;

b = breadth of rectangular beam or breadth of rib of T-beam;

and a = arm of resistance moment.

(ii) Where at any cross-section the value of the shear stress, as calculated from equation (1) above, does not exceed the permissible shear stress for plain concrete given in Clause 301, no shear reinforcement need be provided.

(iii) Where at any cross-section the value of the shear stress, as calculated from equation (1) above, exceeds the permissible shear



M_x AND M_y = BENDING MOMENTS ON STRIPS OF UNIT WIDTH AND SPANS l_x AND l_y RESPECTIVELY.

w = LOAD PER UNIT AREA

l_y = LONG SIDE

l_x = SHORT SIDE

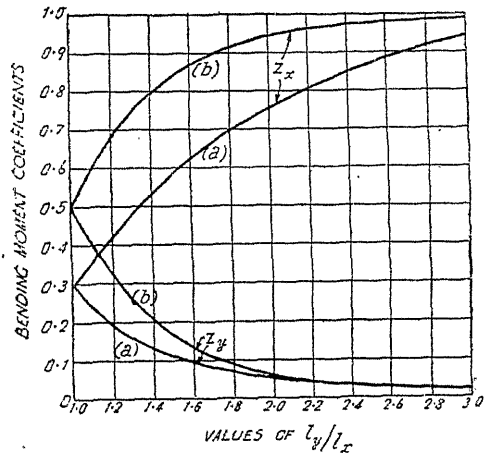


Fig. 478.—*Bending Moment Coefficients for Slabs.*

stress for plain concrete, the whole shear at such cross-section shall be provided for by the tensile resistance of the shear reinforcement acting in conjunction with the diagonal compression of the concrete in the web. In no case shall the shear stress calculated from equation (1) exceed four times the permissible shear stress for plain concrete.

(*b*) *Shear Reinforcement.*—(i) The shear or web reinforcement shall pass round the tensile reinforcement or be otherwise secured thereto and shall be effectively anchored at both ends in such a manner that its full working stress can be developed.

(ii) Tensile reinforcement which is inclined across the neutral plane of a beam and which is carried through a depth equal to the arm of the resistance moment may be taken as shear or web reinforcement, providing it is effectively anchored.

(iii) Where two or more types of web reinforcement are used in conjunction, the total shearing resistance of the beam shall be assumed as the sum of the shearing resistances computed for the various types separately.

(iv) *Stirrups.*—The spacing of stirrups when required under Clause 603 (*a*) (iii) above shall not exceed a length equal to the arm of the resistance moment. The resistance to shear " S " shall then be calculated from equation (2).

$$S = \frac{t_w A_w a}{p} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (2)$$

where t_w = permissible tensile stress allowed in shear or web reinforcement,

A_w = cross-sectional area of stirrup,

p = pitch or spacing of stirrups,

and a = arm of resistance moment.

(v) *Bent-up Bars.*—The resistance to shear at any section of a beam reinforced with bent-up bars may be calculated on the assumption that the bent-up bars from the tension members of one or more single systems of [illegible] which the concrete forms the inclined compression [illegible] shear resistance at any vertical section shall then be taken as the sum of the vertical components of the inclined tension and compression forces cut by such section.

Where on these assumptions the force required for equilibrium in the horizontal portion of a bar is greater than the force in the inclined portion (that is, wherever the angle between the inclined compression and the longitudinal axis of the beam is less than half the angle between the inclined and horizontal portions of the bar), the force in the inclined bar shall be limited to a value such that the permissible stress in the steel is not exceeded in the horizontal portion.

Bond and Anchorage.

604.—(*a*) When reinforcement in the form of plain bars is used to resist tensile stresses induced by bending, the bond stress " s_b " calculated from equation (3) shall not exceed twice the appropriate permissible bond stresses given in Clause 301.

$$s_b = \frac{S}{ao} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (3)$$

where S = total shear across the section,

a = arm of resistance moment,

and o = sum of the perimeters of the bars in the tensile reinforcement of a member.

In members of other than uniform depth the effect of the change in depth on the bond stress shall also be taken into account.

(*b*) Exclusive of a hook or other end anchorage, a bar in tension shall extend from any section for a distance such that the product of the permissible bond stress given in Clause 301, the perimeter of the bar and the length measured from such section is at least equal to the tension required in the bar. In the case of simply supported ends of beams and slabs at least one-quarter of the main tensile reinforcement shall extend to the centre line of the support before the hook or other end anchorage begins. Under the conditions of loading specified in Clause 602 (*a*), at least one-quarter of the tensile reinforcement shall be carried for a distance not less than one-half the effective depth of the beam or slab beyond points of contra-flexure before the hook or other end anchorage begins.

(*c*) A hook at the end of a bar shall be of ⊃ form and shall have an inner diameter of at least four times the diameter of the bar; except that when the hook fits over a main reinforcing bar the diameter of the hook may be equal to the diameter of such bar. The length of the straight part beyond the end of the curve to the end of the hook shall be at least four times the diameter of the bar forming the hook. Unless suitable wrapping or other reinforcement is provided, the anchorage value of the hook shall not be taken into account if the hook is employed in a place where there is a danger of splitting the concrete.

(*d*) When a hook is not used, the end anchorage shall consist of a length of bar or any combination of suitable attachment and length of bar, having an anchorage value equivalent to the resistance produced by the permissible bond stress acting over a length of bar equal to 14 bar diameters. The anchorage value assumed shall be such that neither the permissible stress on the concrete in direct compression nor the safe load on the end anchorage itself is exceeded. The permissible stress on the concrete may be increased to three times the value permitted for the concrete in direct compression where the end anchorage is employed in a place where either the cover of the concrete is sufficient or suitable wrapping or other reinforcement is provided to prevent local failure of the concrete.

(*e*) For flat slabs see Clause 806 (*e*).

SECTION 7—DESIGN—COLUMNS

General Clauses.

701.—(*a*) *Longitudinal Reinforcement.*—The cross-sectional area of longitudinal reinforcement in a column shall be not less than 0·8 per cent. nor more than 8·0 per cent. of the gross cross-sectional area of the column.

Columns with spiral (helical) reinforcement shall have at least six bars within and around the spiral. All other columns shall have one longitudinal bar near each angle point of the column.

At all joints in longitudinal reinforcement the bars shall be overlapped for a length equal to twenty-four times the diameter of the upper bar or a sufficient distance to develop the force in the bar by bond, whichever is the lesser, unless they are otherwise efficiently jointed by welding, screwing, or other means, in such a manner as to develop the full force in the bar.

The limits of bar sizes shall be in accordance with Clause 503.

(*b*) *Transverse Reinforcement.*—(i) *General.*—The volume of transverse reinforcement shall not be less than 0·4 per cent. of the gross volume of the column. Transverse reinforcement shall be so disposed that every longitudinal bar is held against outward buckling, and shall have its ends anchored.

Where adequate restraint is afforded to the main longitudinal reinforcing bars by beams and slabs at points of junction with the columns, the transverse reinforcement may be modified.

The limits of bar sizes shall be in accordance with Clause 503.

(ii) *Lateral Ties.*—The pitch of lateral ties shall not exceed twelve inches and need not be less than six inches. Within these limits the pitch shall not exceed the least lateral dimension of the column or twelve times the diameter of any longitudinal bar.

Where the pitch is the maximum [illegible] of a lateral tie shall be at least one-quarter [illegible] longitudinal bar secured by it. Where a closer pitch is used the diameter of the ties may be reduced provided that the volume of lateral reinforcement is maintained.

(iii) *Spiral (Helical) Reinforcement.*—Spiral reinforcement shall consist of evenly spaced spirals, and shall have its ends anchored.

The pitch of the spirals shall not be more than three inches or one-sixth of the diameter of the core, whichever is the lesser, and shall not be less than one inch, or three times the diameter of the bar composing the spiral, whichever is the greater.

Permissible Loads.

702.—(*a*) *Axially-loaded Columns.*—(i) *Short Columns.*

With Lateral Ties.—The axial load " P " on short columns reinforced with longitudinal bars and lateral ties shall not be greater than the value obtained from equation (4).

$$P = cA_c + tA \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (4)$$

where c = permissible direct stress for concrete given in Clause 301,

t = permissible stress for longitudinal steel in direct compression given in Clause 302,

A_c = cross-sectional area of concrete not including any finishing material applied after the casting of the column,

and A = cross-sectional area of longitudinal steel.

With Spiral Reinforcement.—Where spiral reinforcement is used, the axial load " P " on the column shall not exceed the value given by equation (4) or equation (5) below, whichever is the greater.

$$P = cA_k + tA + 2{\cdot}0\, t_b A_b \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (5)$$

where A_k = cross-sectional area of concrete in the core,

t_b = permissible stress in tension in spiral reinforcement given in Clause 302,

and A_b = equivalent area of spiral reinforcement (volume of spiral per unit length of the column).

In no case shall the sum of the loads contributed by the concrete in the core and by the spiral exceed $0{\cdot}50u\, A_c$ where " u " is the crushing strength of the concrete required from the works test given in Clause 301.

(ii) *Long Columns.*—The permissible working loads of axially loaded long columns shall not exceed the values calculated according to the methods given in Clause 702 (*a*) (i) multiplied by the following buckling coefficients.

Ratio of Effective Length to Least Lateral Dimension of Column.*	Ratio of Effective Length to Least Radius of Gyration.*	Coefficient.
15	50	1·0
18	60	0·9
21	70	0·8
24	80	0·7
27	90	0·6
30	100	0·5
33	110	0·4
36	120	0·3
39	130	0·2
42	140	0·1
45	150	0

* [illegible] on the core a[illegible] taken to be [illegible]

The least lateral dimension of the column may be used only when the cross-section of the column is symmetrical in form about each of two axes at right angles to each other and has no re-entrant angles.

The effective length to be assumed in determining the permissible working load should be as follows :—

	Type of Column.	Effective Column Length.
Columns of one storey	Adequately restrained at both ends in position and direction.	0·75 of the column length.
	Adequately restrained at both ends in position but not in direction.	The column length.
	Adequately restrained at one end in position and direction and imperfectly restrained in both position and direction at the other end.	A value intermediate between the column length and twice that length depending upon the efficiency of the imperfect restraint.

	Type of Column.	Effective Column Length.
Columns continuing through two or more storeys.	Adequately restrained at both ends in position and direction.	0·75 of the column length.
	Adequately restrained at both ends in position and imperfectly restrained in direction at one or both ends.	A value intermediate between 0·75 and 1·00 of the column length, depending upon the efficiency of the directional restraint.
	Adequately restrained at one end in position and direction and imperfectly restrained in both position and direction at the other end.	A value intermediate between the column length and twice that length, depending upon the efficiency of the imperfect restraint.

NOTE.—The effective length values given above are in respect of typical cases only, and er[illegible] the appropriate value :

(*b*) *Bending in Columns.*—Bending moments in internal columns supporting an approximately symmetrical arrangement of beams need not be calculated.

Bending moments in external columns shall be provided for and, unless more exact estimates are made, may be assumed to have the following values :—

	Frames of One Bay.	Frames of Two or More Bays.
Moment at Foot of Upper Column	$M_e \frac{K_u}{K_l + K_u + \frac{K_b}{2}}$	$M_e \frac{K_u}{K_l + K_u + K_b}$
Moment at Head of Lower Column *	$M_e \frac{K_l}{K_l + K_u + \frac{K_b}{2}}$	$M_e \frac{K_l}{K_l + K_u + K_b}$

* These expressions may be used for top storeys by putting $K_u = 0$.

where M_e = bending moment at the end of the beam framing into the external column, assuming both ends of the beam encastered or fixed.
K_b = stiffness of beam,
K_l = stiffness of lower column,
K_u = stiffness of upper column.

The bending moments in columns supporting flat slabs shall be allowed for in accordance with Clause 811 (*b*).

The stiffnesses of members shall be estimated in accordance with Clause 506.

In long columns the maximum stresses shall not exceed the permissible values for short columns, multiplied by the coefficients given in Clause 702 (*a*) (ii).

SECTION 8—FLAT SLABS

Scope.

801.—(*a*) The provisions of the section shall apply only to the design of a series of rectangular slabs of approximately uniform thickness arranged in three or more rows in each direction, and in which the ratio of the length of panel to its width does not exceed one and one-quarter.

(*b*) The length and/or width of any one panel in a series of panels of unequal lengths and/or widths shall not vary from the mean of such lengths or widths by more than 1/40th except at the end spans, which may be shorter.

(*c*) The provisions of this section shall apply to the design of slabs with drops provided that either

(i) the drops shall be square or rectangular in plan and shall have a length in each direction of not less than one-third nor greater than one-half of the panel length in that direction, or

(ii) the drops shall be continuous between columns and of a width not less than one-third nor greater than one-half of the panel width.

Division of Panels.

802.—(*a*) For the purpose of design a flat slab panel shall be considered as divided in each direction into a column strip and a middle strip, as defined in Clause 103 (see Fig. 479).

(*b*) The moments given in Clause 803 (*a*) shall apply to the following sections (see Fig. 479) :—

(i) Positive moment sections along the centre lines of the panel.

(ii) Negative moment sections along the edges of the panel on lines joining the centres of the columns and around the perimeter of the column heads.

(*c*) The width of the column strip shall be taken as equal to one-half the width of the panel excepting that where drops are used it may be taken as equal to the width of the drop.

(*d*) The width of the middle strip shall be taken as equal to one-half of the width of the panel, excepting that where drops are used

and the column strip is taken as equal to the width of the drop the width shall be taken as the difference between the panel width and the drop width.

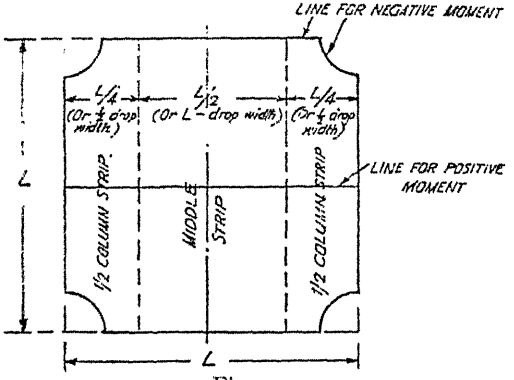


Fig. 479.

Moments in Interior Panels fully Continuous.

803.—(*a*) The moments to be provided for at the sections referred to in Clause 802 (*b*) shall be as follows :—

(i) Panels without drops :—

	Positive Moment.	*Negative Moment.*
Column Strip ...	$0{\cdot}022wL\,(L - \frac{2}{3}D)^2$	$0{\cdot}042wL\,(L - \frac{2}{3}D)^2$
Middle Strip ...	$0{\cdot}018wL\,(L - \frac{2}{3}D)^2$	$0{\cdot}018wL\,(L - \frac{2}{3}D)^2$

(ii) Panels with drops :

	Positive Moment.	*Negative Moment.*
Column Strip ...	$0{\cdot}022wL\,(L - \frac{2}{3}D)^2$	$0{\cdot}046wL\,(L - \frac{2}{3}D)^2$
Middle Strip ...	$0{\cdot}016wL\,(L - \frac{2}{3}D)^2$	$0{\cdot}016wL\,(L - \frac{2}{3}D)^2$

In these and subsequent formulæ throughout Section 8 L represents the length of the panel measured from centre line to centre line of columns, *w* the total dead load plus superimposed load per unit area (see Clause 809 (*a*) for requirements for partitions), and D the diameter of the column head. The formulæ give moments on the whole width of the strip.

(*b*) Where the column strip is taken as equal to the width of the drop, and the middle strip is thereby increased in width to a value greater than half the width of the panel, the moments to be taken on the middle strip shall be increased in proportion to its increased width. The moments to be taken by the column strip may then be decreased by an amount such that there is no reduction in either the total positive or the total negative moments taken by the column and middle strips.

(*c*) In the case of panels other than square panels and within the limits imposed by Clause 801 (*a*), the value of L to be taken in Clause 803 (*a*) shall be the greater of the two dimensions.

Width of Reinforcing Bands.

804.—(*a*) For slabs reinforced in two directions only, the reinforcement in each strip shall be so disposed that the strip is reinforced over its full width.

(*b*) For slabs reinforced in four directions, the width of the direct bands of reinforcement shall be 0·4 times the panel width at right angles to the direction of the reinforcement and the width of the diagonal bands 0·5 times the panel length or 0·5 times the average panel length in the case of panels which are not square.

(*c*) In four-way systems, the reinforcement shall be apportioned as follows :—

(i) The reinforcement in the direct band shall take the entire positive moment in the column strip.

(ii) The reinforcement in the diagonal band shall take the entire positive moment for the middle strip.

(iii) The reinforcement in the direct band, plus the reinforcement in the diagonal bands, the effective area of which shall be calculated as defined in Clause 103, shall take the negative moment in the column strip.

(iv) Additional reinforcement shall be provided to take the negative moment in the middle strip.

Thickness of Slab and Drop.

805.—(*a*) The effective depth of the slab and drop, where used, shall be determined from considerations of bending and shear in accordance with Clauses 806 (*a*) and 810, provided that the effective area of tensile reinforcement used for the calculations is not more than 1 per cent. of the product of the effective depth (which for the purpose of estimating this percentage may be taken as 1 inch less than the total thickness of the slab or drop) and the width of the strip or drop. The total thickness of the slab shall in no case be less than the greatest of the following values :—

(i) 5 inches.

(ii) L/32 for end panels without drops.

(iii) L/36 for interior panels, fully continuous, without drops and for end panels with drops.

(iv) L/40 for interior panels, fully continuous, with drops.

(*b*) The thickness of the drop, measured from the upper surface of the slab, shall not be less than 25 per cent. nor more than 50 per cent. greater than the thickness of the slab.

Arrangement of Reinforcement.

806.—(*a*) Each strip shall at all sections be capable of resisting the moments specified without the use of steel in compression, except in side and end panels as permitted under Clause 807 (*a*) (i) and where openings in panels necessitate rearrangement of reinforcement under Clause 808.

(*b*) *Two-Way Systems of Reinforcement.*—(i) In each strip or band one-half of the positive reinforcement shall be continuous in the lower part of the slab, and shall extend to within a distance of 0·125L measured from the line joining the column centres.

(ii) In each strip or band one-half of the positive reinforcement shall be bent up to form negative reinforcement where such is required. These bars shall extend in the top of the slab into adjacent

panels for an average distance measured from the line joining the column centres of not less than 0·25L, and in no case for a distance less than 0·2L.

(iii) The location of the bends referred to in Clause (*b*) (ii) shall be such that the full area of negative moment reinforcement is provided for a distance measured from the line joining the column centres of not less than 0·2L. The location of the bends shall also be such that the full area of positive moment reinforcement shall be provided for a distance measured from the centre line of the panel of not less than 0·25L.

(*c*) *Four-Way Systems of Reinforcement.*—(i) For direct bands the provisions given in (*b*) above shall apply.

(ii) In each diagonal band one-half of the positive reinforcement shall be [illegible] lower part of the slab and shall extend to within a distance of 0·2L measured from a line drawn through the column centre at right angles to the direction of the band.

(iii) In each diagonal band at least one-half of the positive reinforcement shall be bent up to form negative reinforcement where such is required. These bars shall extend in the top of the slab into adjacent panels for an average distance of 0·4L beyond a line drawn through the column centre at right angles to the direction of the band, and in no case less than 0·35L.

(iv) In each diagonal band the location of bends referred to in Clause (iii) above shall be such that the full area of negative reinforcement is provided for a distance not less than 0·3L measured from a line drawn through the column centre at right angles to the direction of the band. The location of the bends shall also be such that the full area of positive reinforcement is provided for a distance of not less than 0·35L measured from a line drawn through the centre of the panel at right angles to the direction of the band.

(v) The additional reinforcement required to take the negative moment in the middle strip shall extend for a distance not less than 0·25L on either side of the line joining the column centres.

(*d*) In all strips the percentage of reinforcement running in the direction of the strips shall not be less than 0·3 per cent. of the product of the width of the strip and the effective depth.

(*e*) Hooks or other forms of end anchorage need not be provided, except as required in Clause 807 (*a*) (iii).

Moments in Side or End Panels not Fully Continuous.

807.—(*a*) In the case of side or end panels in which the slab is not continuous upon one edge or two adjacent edges,

(i) The positive moments to be used for sections parallel to the discontinuous edges (reinforcement perpendicular to the edges) shall be 25 per cent. greater than those given in Clause 803. For this purpose, providing the slab thickness is not less than that of adjacent fully continuous panels, the effective area of steel in tension may exceed the limit imposed by Clause 805 (*a*) and reinforcement may be used in compression where required.

(ii) At the edges the negative moment (reinforcement perpendicular to the edges) in the column strip shall be taken as

not less than 90 per cent. and in the middle strip not less than 60 per cent. of the moments given in Clause 803 (*a*).

(iii) At the edge the positive and negative reinforcement shall extend to within 3 inches of the edge of the panel and be effectively anchored.

(iv) Where end spans are shorter than interior spans (see Clause 801 (*b*) the moments given in (i) and (ii) above may be suitably modified.

(*b*) Slabs which are not continuous over two opposite edges shall be considered for positive moments as freely supported at these edges for the calculation of the moments in a direction at right angles to the freely supported edges. For negative moments Clause (*a*) (ii) above shall apply.

(*c*) (i) In the case of approximately square panels supported by beams upon one edge or upon two adjacent edges, each beam shall be designed to carry at least the direct load upon it plus one-quarter of the total panel load.

(ii) In the case of panels supported by beams upon two opposite edges the beam shall be designed to carry at least the direct load upon it plus one-half of the total panel load.

(*d*) In a half column strip adjacent to an edge beam the reinforcement parallel to the beam need not exceed one-quarter that specified for the column strip in Clause 803 (*a*).

Openings in Panels.

808.—(*a*) Excepting for openings complying with (*b*), (*c*), and (*d*) below, openings shall be completely framed on all sides with beams to carry the loads to the columns.

(*b*) Openings of a size such that the greatest dimension in a direction parallel to a centre line of the panel does not exceed $0{\cdot}4L$ may be cut through the panel in the area common to two intersecting middle strips, provided the total positive and negative moments be maintained as specified in Clause 803 (*a*) and that these total positive and negative moments be redistributed between the remaining principal design sections to meet the new conditions.

(*c*) One or more openings of aggregate length or width not exceeding one-tenth of the width of the column strip may be made in the area common to two column strips provided that the reduced sections are capable of carrying the appropriate moments specified in Clause 803 (*a*).

(*d*) Openings of an aggregate length or width not exceeding one-quarter of the width of the strip may be made in any area common to one-column strip and one middle strip provided that the reduced sections are capable of carrying the appropriate moments specified in Clause 803 (*a*).

Partitions and Walls

809.—(*a*) All partitions and walls whose aggregate weight exceeds one-twentieth of the sum of the dead and superimposed loads shall be carried by beams; these beams shall transmit all loads carried by them to the columns either directly or by means of other beams.

Shearing Stresses.

810.—(*a*) Neither the shearing stress in the slab or drop upon a vertical section at a distance equal to the effective depth from the

column head nor the shearing stress upon a vertical section along the perimeter of the drop, where used, shall exceed the permissible values given in Clause 301.

Columns Supporting Flat Slabs.

311.—(*a*) (i) Interior columns shall be provided with enlarged heads, the diameter (D) of which shall not be less than 0·2L, nor more than 0·25L, except where the column itself is of such diameter.

(ii) The diameter of the column head shall be taken on a plane parallel to and 1½ in. below the underside of the slab or drop, and shall be the diameter intercepted by this plane on the largest inverted circular cone contained entirely within the column and its enlarged head below this plane. The vertex angle of the cone shall be a right angle and its axis shall be the centre line of the column. (See Fig. 480.)

(iii) All exterior wall columns shall be provided with such portion of the enlarged head as specified in Clause (*a*) (ii) above as

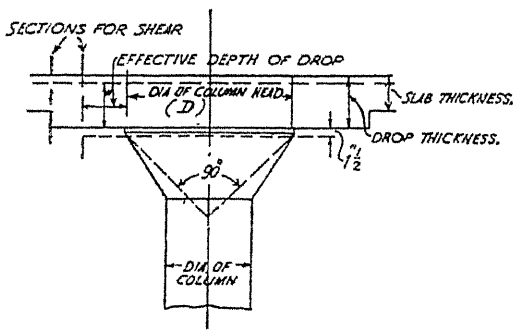


Fig. 480.

will lie within the adjoining walls, or, when rectangular columns are used with beams, the enlarged head may consist of an internal bracket not less than the full width of the inside face of the column.

(iv) The value D for a bracket head in the direction in which the bracket extends may be taken as twice the distance from the centre of the column to a point where the structural portion of the bracket thickness is 1½ in. measured vertically from the underside of the slab or drop to a plane inclined at 45 degrees to the inside face of the column lying entirely within the bracket head, and this value of D averaged with the value of D for an interior column head in the calculations for moment under Clause 803 (*a*). The value of D for column strips parallel and adjacent to a non-continuous edge of a slab where either no [illegible] is used, or where the beam used is not deeper than [illegible] times the minimum slab thickness, should be taken as equal to the width of the wall column if no bracket is provided in this direction.

(v) The value for D for column strips parallel and adjacent to

marginal beams having a depth greater than one and a half times the thickness of the slab at the wall columns, shall, if no bracket is provided in this direction, be taken as equal to the width of the wall column plus twice the difference between the depth of the beam and the depth of the slab at the column head.

(*b*) (i) Moments in internal and in external columns shall be provided for and shall be taken as equal to 50 and 90 per cent. respectively of the negative moment in the column strip, specified in Clause 803 (*a*). This moment shall be apportioned between the upper and lower columns in proportion to their stiffnesses.

(ii) In the case of external columns carrying a portion of the floor and/or walls as a cantilevered load the specified column moments may be reduced by the moment due to the dead load on the cantilevered portion.

SECTION 9—DESIGN—WALLS AND SPECIAL FORMS OF CONSTRUCTION

Reinforced Concrete Walls.

901.—All reinforced concrete walls shall comply with the relevant general building clauses (see Appendix (I) of this Code of Practice) and shall be of such design and construction as to ensure a standard of strength equivalent to that demanded for other structural members.

Special Forms of Construction.

902.—Special forms of construction not otherwise provided for in this Code of Practice may be employed where the methods of design and construction are such as to ensure a standard of strength and durability at least equivalent to that demanded by this Code of Practice.

Special Materials.

903.—Special materials having special properties, including deformed bars, may be employed at their appropriate stresses where the standard of strength and durability is at least equivalent to that demanded by this Code of Practice.

SECTION 10—TESTS OF STRUCTURES OR STRUCTURAL ELEMENTS

1001.—Loading tests on a completed structure or structural member may be demanded in all cases where there is reasonable doubt as to the adequacy of the strength of any part of the structure. Such tests need not be made until the expiry of 56 days of effective hardening of the concrete.

In such tests, the structural member under consideration shall be subjected to a superimposed load equal to one and a half times the specified superimposed load used for design, and this load shall be maintained for a period of 24 hours before removal.

If during the test, or upon removal of the load, the member shows signs of weakness or faulty construction, it shall be reconstructed or strengthened in accordance with this Code of Practice.

If after the removal of the load the slabs or beams do not show a recovery of at least 75 per cent. of the maximum deflection shown during the 24 hours under load, the test loading shall be repeated.

The structure shall be considered to have failed to pass the test if the recovery after the second test is not at least 75 per cent. of the maximum deflection shown during the second test.

APPENDIX I
GENERAL BUILDING CLAUSES

1.—

Definitions.

(*Note.*—In order to make this Appendix complete in itself some definitions already given in the body of the Code are repeated here.)

Beam. A member primarily carrying transverse loads by bending, but not of considerable width relative to its thickness.

Bearing Wall or Bearing Structure. A wall or structure which provides support for other structural members, and which is of sufficient strength and stability adequately to carry its own weight together with all imposed loads and forces.

Column. A structural member primarily carrying axial load by means of compression stresses.

Dead Load of a Building. The actual weight of all permanent construction comprised in the building.

Effective Column Length. The length upon which the ratio of column length to least lateral dimension is calculated in Clause 702 (*a*) (ii).

External Wall. An outer wall of a building, not being a party wall, even though adjoining a wall of another building.

Foundation. A structure entirely below the level of the ground, which is employed for the purpose of distributing the load from columns, beams or walls on to the ground, and may include any retaining or other wall based upon the ground, provided that it is of sufficient strength and stability adequately to carry its own weight together with all imposed loads and forces.

Grade of Fire-Resistance. As defined in relation to British Standard Specification No. 476.

Mass Concrete. Concrete for which the aggregate is measured as combined aggregate.

Non-Bearing Wall or Non-Bearing Structure. A wall or structure which supports no load other than its own weight and any local wind pressure which may act upon its surface.

Panel Wall. A non-bearing external wall built between columns and wholly supported by beams, foundations or bearing structures.

Partition. An internal vertical structure employed solely for the purpose of sub-dividing any storey of a building into sections, and which supports no load other than its own weight.

Party Wall. (*a*) A wall forming part of a building and used or constructed to be used in any part of its height or length for the separation of adjoining buildings; or (*b*) wall forming part of a building and standing in any part of its length to a greater extent than the projection of the footings on one side, on ground of different owners.

Structural Members or Structural Framework. Any beams, slabs or columns or assemblage of beams, slabs and/or columns provided for the purpose of supporting any portion of the load of the building or of resisting any forces imposed upon it.

Superimposed Load. All load other than dead load.

General. 2.—The structural members and/or structural framework of a building in combination with the floors, party walls, bearing walls and bearing structures (if any) and the foundations shall be capable of sustaining safely, and without exceeding the limits of stress hereinafter specified, the whole dead and superimposed load of the building together with all forces due to wind, earth or other pressures acting upon it.

Where a British standard specification has been issued, the stipulations have generally been embodied in this Code. Such modifications as have been made in certain clauses are to be regarded as superseding, for the purposes of this Code, the existing British standard specification.

Where no British standard specification has yet been issued, the standards defined in this Code are to serve as a practical guide.

Fire Protection. 3.—For all buildings which are required to be of a specified degree of fire resistance (with respect to height, occupancy, position and other relevant factors), the grade of fire resistance of the elements of structure (walls, floors, partitions, doors, etc.) and the incombustible and non-inflammable properties of materials shall be stipulated in accordance with British standard specification for fire resistance, incombustibility and non-inflammability of building materials and structures No. 476.

MATERIALS

Cement and Lime. 4.—All cement shall comply with the requirements given in Clause 201 of this Code of Practice.

All lime shall comply with the British standard specification for building mortar (lime).

Sand for Mortars and Combined Aggregate for Mass Concrete. 5.—All sand and combined aggregate shall be sound, strong, clean, free from harmful impurities, and suitable for its required purpose.

All combined aggregate for mass concrete shall be composed of approved stone, ballast or hard well-burnt broken brick passing a 2-in. ring and containing sufficient sand to produce a dense concrete with no considerable excess of sand.

Fine and Coarse Aggregate for Reinforced Concrete.

6.—All fine and coarse aggregate for reinforced concrete shall comply with Clause 202 of this Code of Practice.

Cement Mortars.

7.—Cement mortar shall be mixed in one of the following or intermediate proportions :—

Specification of Mortar.	Composition.	
	lb. of Cement.	Cubic Feet of Clean Silicious Sand.
1 : 2 cement mortar ...	112	$2\frac{1}{2}$
1 : 3 cement mortar ...	112	$3\frac{3}{4}$
1 : 4 cement mortar ...	112	5

Lime Mortars.

8.—All lime mortar mixes shall be such as will set in 48 hours, and the set mortar shall be sound and of good strength.

All lime mortars shall be mixed in one of the following or intermediate proportions :—

Specification of Mortar.	Composition.	
	Cubic Feet of Properly Slaked Lime of Usual Consistence or Dry Hydrate.	Cubic Feet of Clean Silicious Sand or other Approved Aggregate.
1 : 2 lime mortar	1	2
1 : 3 lime mortar	1	3

Lime Mortars Gauged with Cement.

9.—Lime mortars may be gauged with cement. The volume of cement shall not be less than one-fifth of the volume of lime and may be considered as replacing part of the lime in the table of proportions given in Clause 8 for Lime Mortars.

Concrete.

10.—All concrete shall be mixed in proportions complying with Clause 301 of this Code of Practice excepting mass concrete which shall be mixed in one of the following or intermediate proportions :—

Specification of Concrete.	Composition.	
	Lb. of Cement.	Cubic Feet of "Combined Aggregate" (see Clause 5).
1 : 6 Mass Concrete... ...	112	7½
1 : 8 Mass Concrete... ...	112	10
1 : 10 Mass Concrete ...	112	12½
1 : 12 Mass Concrete ...	112	15

Concrete containing breeze or clinker aggregate shall not be used in any floor, wall, bearing structure or foundation except that in the case of panel walls clinker aggregate may be used for pressed bricks or pre-cast blocks when the aggregate is weathered and the bricks or blocks matured to the satisfaction of the designer, and it is shown to his satisfaction and by the tests given in Appendix (IX) to be sufficiently free from unburnt coal, sulphur and other deleterious substances to produce a sound durable concrete of the required strength. In no case shall such aggregate be in any position within 1 in. of any structural steel or reinforcement.

The materials employed for all concrete and the methods employed in its mixing and deposition shall be such that the finished concrete has a crushing strength at 28 days of not less than three times the maximum permissible load given in Clause 19, and a crushing strength at any less age of at least three times the load that it may be called upon to carry at that age.

Hollow Blocks.

11. All hollow blocks shall be made of concrete, well burnt brick earth or well burnt clay, and shall comply with the tests given below.

The aggregate width of voids in any block measured horizontally at right angles to the face of the block as laid in the wall shall not exceed two-thirds of the total thickness of the block, and the net volume of material in any block shall not be less than one-half of its gross volume.

Hollow blocks with an aggregate width of voids (measured horizontally at right angles to the face of the block as laid in the wall) of two-thirds the total thickness of the block shall have a minimum crushing strength of 450 lb. per square inch calculated on the gross area of the block.

For hollow blocks having an aggregate width of voids (measured as above) of less than two-thirds of the total thickness of the block,

the minimum crushing strength shall be increased in the proportion of :—

$$3 \times \left(\frac{\text{thickness of block—width of voids}}{\text{thickness of block}}\right)$$

To ascertain the crushing strength twelve blocks shall be tested with the cells placed at right angles to the direction of application of the load and the mean of the results taken. The blocks shall be soaked in water at 15°—20° C. for 24 hours. The top and bottom surfaces shall then be made plane and parallel by the application of a 1 : 3 cement mortar rendering, the mortar being applied in such a thickness that in no part shall it amount to less than ¼ in. clear of the block. The cement used shall be either normal or rapid-hardening Portland cement. After the application of the mixture to the blocks the latter shall be covered with damp cloths for 24 hours and then immersed in water for a further period until tested between plywood sheets. For normal Portland cement the period of immersion shall be 27 days, and for rapid-hardening Portland cement 6 days, or such period, not less than 2 days, at which 3-in. mortar cubes made from the batch of mortar used in the preparation of the blocks and stored under identical conditions shall have a crushing strength between 2,000 and 4,000 lb. per sq. in.

Bricks.

12.—The minimum crushing strength of bricks, in pounds per square inch, shall be :—

For bricks of first strength	... ...	10,000
,, second strength	... ...	5,000
,, third strength	... ...	3,000
,, fourth strength	... ...	1,500

To ascertain the crushing strength twelve bricks shall be tested and the mean of the results taken. All frogs shall be filled as laid in the wall with 1 : 3 cement mortar. The cement used and the conditions and periods of storage both before and after filling the frogs, shall comply with those specified in Clause 11 for the testing of hollow blocks.

Details of Construction

Party and Bearing Walls, etc.

13.—(*a*) *General.*—All party walls, bearing walls, bearing structures and foundations shall be built of bricks, stones, concrete, steel or a combination thereof, or of other approved materials which provide adequate strength and durability together with an adequate grade of fire resistance. All bricks and stone work shall be properly bonded and solidly bedded and jointed throughout.

(*b*) *Party Walls.*—Party walls may be constructed as panel walls of solid brick or reinforced concrete supported by a structural framework, subject to the following provisions :—

(i) *For brickwork party wall panels.*—All brickwork panels shall be of at least 13 inches thickness throughout and the least

span of a panel vertically or horizontally between its structural supports shall in no case exceed sixteen times its thickness.

(ii) *For reinforced concrete party wall panels.*—All reinforced concrete panels shall be cast *in situ* of 1 : 2 : 4 (or richer mix) cement concrete properly reinforced with steel, shall be of at least 8 inches solid thickness throughout exclusive of rendering plaster or other decorative finish, and their reinforcement shall be adequately anchored to the structural framework.

Panel Walls.

14.—(*a*) *General.*—All panel walls shall be constructed of bricks, stones, hollow blocks, concrete or a combination thereof, or of other approved materials which provide adequate strength and durability together with an adequate grade of fire resistance. All panel walls shall be so built as to be [illegible]red to the structural framework, and no panel shall [illegible] height between supporting beams than 25 feet. All bricks, stones, hollow blocks or other such units shall be properly bonded, and bedded and jointed in mortar with all joints full and solidly made.

(*b*) *Unfaced panel walls of one thickness of bricks, stones, etc.*—Panel walls constructed with one single thickness of brick, concrete or hollow block units set and jointed in mortar shall, except as provided in Clause 14 (*f*) hereinafter, be of at least 8½ inches overall thickness throughout exclusive of rendering, plaster or other decorative finish. In cases where the clear span of any such panel between structural supports (either horizontally or vertically, whichever is the lesser) exceeds 13 feet, the thickness thereof shall throughout be increased by at least 2 inches for each 3 feet or fraction thereof of such excess span. Such additional thickness shall not be provided by unduly increasing the width of the mortar joints.

(*c*) *Faced panel walls of one thickness of bricks, stones, etc.*—Panel walls constructed with one single thickness of units, set and jointed in mortar, of bricks or concrete with a facing of terra-cotta, faience or stone, or of hollow blocks with a facing of bricks, terra-cotta faience or stone, shall be of at least 13 inches overall thickness throughout exclusive of rendering, plaster, or other decorative finish. In cases where the clear span of any such panel between structural supports (either horizontally or vertically, whichever is the lesser) exceeds 16 feet, the thickness thereof shall be increased by at least 2 inches for each 3 feet or fraction thereof of such excess span. Such additional thickness shall not be provided by unduly increasing the width of the mortar joints. The brick, terra-cotta, faience or stone facing shall in all cases be either bonded or otherwise adequately secured to the backing.

(*d*) *Cavity panel walls of bricks, stones, etc.*—Panel walls constructed with solid brick concrete or stone units set and jointed in mortar may be formed as cavity walls composed of two solid walls or shells each not less than 4½ inches thick exclusive of rendering, plaster or other decorative finish. Except as otherwise provided hereinafter, the two shells shall be spaced not more than 3 inches apart and shall be securely tied together across the cavity with ties of substantial rust resisting design spaced at the rate of two ties to

every square yard of cavity area and so arranged that their vertical spacing is one-half of their horizontal spacing and that alternate rows of ties are "staggered."

The two shells may be spaced more than 3 inches apart but not more than 6 inches apart provided that the number of ties be increased in direct proportion to the cavity width.

The clear span of any single panel of cavity wall between structural supports (either horizontally or vertically, whichever is the lesser) shall not exceed 13 feet, and the overall area of any single panel (between structural supports) shall not exceed 200 square feet. In cases where this limit of either span or area is exceeded, or where the cavity between the shells exceeds 6 inches, one of the shells shall be made to conform to the requirements for panel walls of one thickness (see Clause 14 (*b*) or (*c*)).

(*e*) *Panel walls of monolithic reinforced concrete.*—Panel walls constructed of reinforced concrete shall be cast *in situ* of 1 : 2 : 4 (or richer mix) cement concrete properly reinforced with steel, shall be of at least 4 inches solid thickness throughout exclusive of rendering, plaster or other decorative finish, and their reinforcement shall be adequately anchored to the structural framing on all four sides.

(*f*) *Panel walls of one thickness of bricks for one storey buildings.*—Panel walls constructed with one thickness of solid bricks set and jointed in cement mortar may be employed in the walls of one storey buildings not used for habitation. Such walls may be not less than 4½ inches thick provided that they are adequately supported by piers, stanchions or beams. The clear [illegible] of any such panel between vertical supports shall not exceed 10 feet, and its clear overall area shall not exceed 120 square feet.

(*g*) *Permissible overhang of panel walls.*—In no case shall a panel wall constructed in accordance with Clause 14 (*b*), (*c*) or (*e*) (or the shell of a panel wall constructed in accordance with Clause 14 (*d*)) overhang the concrete edge of its supporting beam by more than one-third of the thickness of the wall (or shell), unless the beam is reinforced in combination with an adjoining floor slab in such a manner as to transfer the load axially to the beam.

A cavity wall, the base courses of which are built solid for the full thickness of the wall and to a height above its base of not less than the full wall thickness, may overhang the edge of its supporting beam to the same extent as a solid panel wall.

Loads

Superimposed Loads.

15.—(*a*) *Schedule of Loads.*—For the purpose of calculating the loads on slabs, beams, columns, piers, walls and foundation, the minimum superimposed load on each floor and on the roof shall be estimated as equivalent to the following dead loads :—

Class No.	Type of Building or Floor.	Slabs. Lb. per sq. ft. of Floor Area.	Beams, Columns, Piers, Walls and Foundations. Lb. per sq. ft. of Floor Area.
1	Rooms used for domestic purposes, hotel bedrooms, hospital rooms and wards	50	40
2	Offices, floors above entrance floor	80	50
3	Offices, entrance floor and floors below entrance floor	80	80
4	Churches, schools, reading rooms, art galleries and the like ...	80	70
5	Retail shops and garages for cars of not more than two tons dead weight	80	80
6	Assembly halls, drill halls, dance halls, gymnasia, light workshops, public spaces in hotels and hospitals, staircases and landings, theatres, cinemas, restaurants and grandstands	100	100
7	Warehouses, book stores, stationery stores and similar uses, together with garages for motor vehicles exceeding two tons dead weight; actual load to be calculated but not less than	200	$\frac{1p}{c}$ 200
Class No.	Roofs.	Lb. per sq. ft. of Covered Area.	Lb. per sq. ft. of Covered Area.
8	Flat roofs and roofs inclined at an angle with the horizontal of not more than 20°	50	30

On roofs inclined at an angle with the horizontal of more than 20° a minimum superimposed load (deemed to include the wind load) of 15 lb. per square foot of surface shall be assumed acting normal to the surface inwards on the windward side, and 10 lb. per square foot of surface similarly acting outwards on the leeward side, provided that this requirement shall apply only in the design of

the roof structure, and that a vertical applied load of 10 lb. per square foot of covered area shall be substituted for it in estimating the applied roof load upon all other parts of the construction.

Beams and ribs within the depth of the floor but not spaced further apart than 3 feet between centres shall be designed for slab loads.

(*b*) *Partition Loads.*—In all cases where the positions of the partitions are definitely located in the design the actual weight of the partitions shall be included in the dead floor load.

In all cases of floors where the positions of the partitions are not definitely located in the design a uniformly distributed load sufficient to allow for them shall be added to the dead floor load and for all such floors used for offices the minimum total allowance for partitions shall be at the rate of 20 lb. per square foot of floor area.

(*c*) *Alternative Loads on Slabs and Beams.*—Floor slabs and beams shall be capable of carrying the following alternative loads on an otherwise unloaded floor :—

Class of Floor.	Alternative Minimum Superimposed Load.	
	Slabs.	Beams.
Floors scheduled under Class I	¼ ton uniformly distributed per foot width.	1 ton uniformly distributed.
All floors scheduled under Classes 2—7, except garage floors under Class 7	⅜ ton uniformly distributed per foot width.	2 tons uniformly distributed.
Garage floors under Class 7	1·5 × maximum wheel load but not less than 1 ton considered distributed over a floor area of 2 ft. 6 in. square.	

In the case of slabs the alternative superimposed load where specified as per foot width shall be taken on a length equal to the span in the case of slabs spanning in one direction and equal to the shorter span in the case of slabs spanning in two directions at right angles.

The reactions due to these alternative loads need not be allowed for in calculating the loads on columns, piers, walls or foundations.

(*d*) *Column Loads.*—For the purpose of calculating the total load to be carried on columns, piers, walls and foundations in buildings of more than two storeys in height, the superimposed loads for the roof and topmost storey shall be calculated in full in accordance with the schedule of loading, but for the lower storeys a reduction of the superimposed loads may be allowed in accordance with the following table :—

Next storey below topmost storey	10 per cent. reduction of its superimposed load.		
Next storey below	20	do.	do.
Next storey below	30	do.	do.
Next storey below	40	do.	do.
Each succeeding storey	50	do.	do.

The above reduction may be made by estimating the proportion of floor area carried by each foundation, column, pier and wall. No such reductions shall be allowed on any floor scheduled for an applied loading exceeding 100 lb. per square foot.

(*e*) *Special Loads.*—In any case where the superimposed load on any floor or roof is to exceed that hereinbefore specified for such floor or roof, such greater load shall be provided for pursuant to Clause 2 hereinbefore.

In the case of any floor intended to be used for a purpose for which a superimposed load is not specified herein, the superimposed load to be carried on such floor shall be provided for pursuant to Clause 2 hereinbefore.

Wind Pressure.

16.—All buildings other than those indicated below shall be so designed as to resist safely a wind pressure in any horizontal direction of not less than 15 lb. per square foot upon the upper two-thirds of the vertical projection of the surface of such buildings, with an additional pressure of 10 lb. per square foot upon all projections above the general roof level.

If the height of a building is less than twice its width, wind pressure may be neglected, provided that the building is stiffened by floors and/or walls.

Exhibition of Notice of Safe Applied Load.

17.—In every building scheduled for an applied loading exceeding 100 lb. per square foot a notice shall be permanently exhibited in a conspicuous position on every floor stating the total superimposed load per square foot of floor area for which the floor has been designed (inclusive of partitions, if any).

Permissible Pressures upon Subsoil, Brickwork, Concrete and Stonework

Pressure upon Subsoil.

18.—The following permissible loads upon various subsoils are given as a general guide to their safe bearing capacity, but the building owner shall be responsible for providing any trial holes, loading tests or other measures necessary to ascertain the safe bearing load of the ground upon which the founds of a building are to be bedded.

	Permissible Load on Ground. Tons per square foot.
Alluvial soil, made ground, very wet sand... ...	Up to ½
Soft clay, wet or loose sand	Up to 1
Ordinary fairly dry clay, fairly dry fine sand, sandy clay	Up to 2
Firm dry clay...	Up to 3
Compact sand or gravel, London blue or similar hard compact clay...	Up to 4
Hard solid chalk	Up to 6
Shale and soft rock	Up to 10
Hard rock	Up to 20

Intermediate values and values for other materials shall be agreed in consultation with the local building authority.

The above pressures may be exceeded by an amount equal to the weight of the material in which a foundation is bedded and which is displaced by the foundation itself, measured downward from the final finished lowest adjoining floor or ground level.

Pressures on Load Bearing Concrete.

19.—The bearing pressures on load bearing concrete in bearing walls, bearing structures and foundations shall not exceed the following values except in reinforced concrete as provided for in the body of this Code of Practice :—

	Tons. per sq. foot.
1 : 12 mass concrete	5
1 : 10 mass concrete	10
1 : 8 mass concrete	15
1 : 6 mass concrete	20
1 : 2 : 4 concrete	30
1 : 1½ : 3 concrete	35
1 : 1 : 2 concrete	40

The above pressures on mass concrete may be exceeded by an amount up to 20 per cent. in all cases where such increased pressure is of a local nature, as at girder bearings.

Pressure on Load Bearing Brickwork.

20.—The pressure per square foot upon properly bonded and solidly bedded brickwork shall not exceed the following values :—

	Tons per sq. foot.
On bricks of fourth strength set in 1 to 2 lime mortar or 1 to 3 lime mortar gauged with cement	4
On bricks of fourth strength set in 1 to 4 cement mortar	8
On bricks of third strength set in 1 to 4 cement mortar	10
On bricks of second strength set in 1 to 3 cement mortar	15
On bricks of first strength set in 1 to 3 cement mortar...	20

The proportions given above represent the least quantities of cement and/or lime that may be used for the mortar corresponding to the pressures specified.

The above pressures may be exceeded by an amount up to 20 per cent. in all cases where such increased pressure is only of a local nature, as at girder bearings.

In the case of isolated brick piers without proper lateral supports, the above stresses may only be used when the ratio of the height to the least dimension of the pier does not exceed 6. For higher values of the ratio of height to least dimension the permissible pressures shall be those given above multiplied by the following reduction coefficients :—

Ratio of height to least dimension ...	8	10	12
Reduction coefficient...	0·8	0·6	0·4

No isolated brick pier shall have a ratio of height to least dimension greater than 12, nor a least horizontal dimension less than 13 inches.

Pressures upon Stonework.

21.—The permissible pressures on stonework (other than rubble masonry) may be classified as for brickwork according to the crushing strength of the stone, provided that for masonry constructed

of large blocks of good quality dressed stone, the corresponding pressures may be increased up to 50 per cent. above those given in Clause 20 for loading-bearing brickwork.

For isolated stonework piers without adequate lateral supports the permissible stresses shall be reduced by the same ratios given above for ratios of height to least dimension greater than 6.

No isolated stonework pier shall have a ratio of height to least dimension greater than 12, nor a least horizontal dimension less than 13 inches.

APPENDIX II

STANDARD METHOD OF TEST FOR ORGANIC IMPURITIES IN SANDS FOR CONCRETE

This method of test is an approximate method of determining the presence of injurious organic compounds in natural sands which are to be used in cement mortar or concrete. The principal value of the test is to furnish a warning that further tests of the sands are necessary before they are approved for use.

A representative test sample of sand, weighing about one pound, shall be obtained by quartering.

A 12 oz. graduated clear glass bottle shall be filled to the 4½ oz. mark with the sand to be tested. A 3 per cent. solution of sodium hydroxide in water shall be added until the volume of the sand and liquid indicated after shaking is 7 liquid ounces. The bottle shall be stoppered, shaken vigorously and then allowed to stand for 24 hours.

A standard colour solution may be prepared by adding 2·5 c.c. of a 2 per cent. solution of tannic acid in 10 per cent. alcohol to 97·5 c.c. of a 3 per cent. sodium hydroxide solution. This should then be placed in a 12 oz. bottle, stoppered, shaken vigorously, and allowed to stand for 24 hours.

After standing 24 hours, the colour of the clear liquid above the sand shall be compared with the colour of the standard colour solution prepared at the same time and in accordance with the method described above, or with glass or other suitable standard of a colour similar to that of the standard solution.

APPENDIX III (A)

STANDARD METHOD OF DECANTATION TEST FOR SAND AND OTHER FINE AGGREGATES

This method of test covers the determination of the total quantity of silt, loam, clay, etc., in sand and other fine aggregates.*

The sample must contain sufficient moisture to prevent segregation and shall be thoroughly mixed. A representative portion of the sample sufficient to yield approximately 500 grams of dried material, shall then be dried to a constant weight at a temperature not exceeding 110° C. (230° F.).

The dried material shall be placed in a pan of approximately 9 inches (230 mm.) in diameter, and not less than 4 inches (102 mm.)

in. depth, and sufficient water added to cover the sample (about 225 c.c.). The contents of the pan shall be agitated vigorously for 15 seconds, and then be allowed to settle for 15 seconds, after which the water shall be poured off, care being taken not to pour off any sand. This operation shall be repeated until the wash water is clear. As a precaution, the wash water shall be poured through a B.S. No. 200-mesh sieve and any material retained thereon returned to the washed sample. The washed sand shall be dried to a constant weight at a temperature not exceeding 110° C. (230° F.).

The results shall be calculated from the formula :—

Percentage of silt, clay, loam, etc. =

$$\frac{\text{Original dry weight—weight after washing}}{\text{Original dry weight}} \times 100.$$

When check determinations are desired, the wash water shall be evaporated to dryness, the residue weighed and the percentage calculated from the formula :—

Percentage of silt, loam, clay, etc. =

$$\frac{\text{Weight of Residue}}{\text{Original dry weight}} \times 100$$

* This determination of the percentage of silt, clay, loam, etc., will include all water-soluble material present, the percentage of which may be determined separately as desired.

APPENDIX III (B)

PRELIMINARY FIELD METHOD OF DECANTATION TEST FOR SAND AND OTHER FINE AGGREGATES

This method of test is intended as a guide to the quantity of silt, loam, clay, etc., and where an aggregate passes the test it shall be deemed unnecessary to carry out the standard test given in Appendix III (A).

A sample of the sand to be tested shall be placed in a 200 c.c. measuring cylinder filling it up to the 100 c.c. mark.

Clean water shall be added up to the 150 c.c. mark.

The mixture shall be shaken vigorously and the contents allowed to settle for one hour.

The volume of silt visible at the surface of the sand shall be noted and recorded as the percentage volume of silt in the sand.

The aggregate shall be deemed satisfactory when this percentage volume does not exceed 6 per cent.

APPENDIX IV

STANDARD METHOD OF SIEVE ANALYSIS OF AGGREGATES FOR CONCRETE

A representative test sample of the aggregate as received on the works shall be selected by quartering. It shall weigh not less than :—

(*a*) Fine aggregate, 3 lb.

(*b*) Coarse aggregate, 10 lb.

(*c*) Where fine and coarse aggregates are to be used combined, 15 lb.

The sample shall be dried to constant weight at a temperature not exceeding 110° C. (230° F.).

The sieves shall be of square-mesh wire-cloth and shall be mounted on substantial frames constructed in a manner that will prevent loss of material during sieving. The size of wire and sieve openings shall conform to the B.S.S. for sieves.

Unless otherwise stated the sample shall be separated into a series of sizes by means of the following sieves, 1½-inch, ¾-inch, ⅜-inch, $\frac{3}{16}$-inch No. 7, No. 14, No. 25, No. 52 and No. 100. Sieving shall be continued until not more than 1 per cent. by weight of the residue passes any sieve during one minute. Each size shall be weighed on a balance or scale which is sensitive to 0·001 of the weight of the test sample.

The percentage by weight of the total sample which is finer than each of the sieves shall be computed.

APPENDIX V

STANDARD METHOD OF TEST FOR YIELD POINT OF STEEL

When the permissible stress is dependent upon the yield point stress of the steel, this stress shall be determined by tests on bars the full size as used and shall be calculated on the nominal area of the cross-section of the bar.

Direct observations of the movements of the bar shall be made to determine the yield point stress. For steels which have no well-defined yield point the yield point stress shall be taken as the stress at which the permanent set of the bar reaches a value of 0·2 per cent. of the original gauge length. Such permanent set may be taken as the amount of deviation from the initial stress-strain proportionality.

APPENDIX VI

STANDARD METHOD OF TEST FOR CONSISTENCE OF CONCRETE

The test is to be used both in the laboratory and during the progress of the work for determining the consistence of concrete.

The test specimen shall be formed in a mould in the form of the frustrum of a cone with internal dimensions as follows :—Bottom diameter, 8 inches ; top diameter, 4 inches, and height, 12 inches. The bottom and the top shall be open, parallel to each other, and at right angles to the axis of the cone. The mould shall be provided with suitable foot pieces and handles. The internal surface shall be smooth.

Care shall be taken to ensure that a representative sample is taken.

The internal surface of the mould shall be thoroughly clean, dry and free from set cement before commencing the test.

The mould shall be placed on a smooth, flat, non-absorbent surface, and the operator shall hold the mould firmly in place, while it is being filled, by standing on the foot pieces. The mould shall be filled to about one-fourth of its height with the concrete which shall then be puddled, using 25 strokes of a §-inch rod, 2 feet long, bullet pointed at the lower end. The filling shall be completed in successive layers similar to the first and the top struck off so that the mould is exactly filled. The mould shall then be removed by raising vertically, immediately after filling. The moulded concrete shall then be allowed to subside and the height of the specimen measured after coming to rest.

The consistence shall be recorded in terms of inches of subsidence of the specimen during the test, which shall be known as the slump.

APPENDIX VII

STANDARD METHOD OF MAKING PRELIMINARY CUBE TESTS OF CONCRETE

The method described applies to compression tests of concrete made in a laboratory where accurate control of materials and test conditions is possible.

Materials and Proportioning.—The materials and the proportions used in making the preliminary tests shall be similar in all respects to those to be employed in the work. The water content shall be as nearly as practicable equal to that to be used in the work, but shall be not less than the sum of 30 per cent. by weight of the cement and 5 per cent. by weight of the aggregate unless specially authorised by the Designer. For porous aggregates additional water shall be used to allow for the amount absorbed by the aggregates.

Materials shall be brought to room temperature (58° to 64° F.) before beginning the tests. The cement on arrival at the laboratory shall be mixed dry either by hand or in a suitable mixer in such a manner as to ensure as uniform a material as possible, care being taken to avoid intrusion of foreign matter. The cement shall then be stored in air-tight containers until required. Aggregates shall be in a dry condition when used in concrete tests.

The quantities of cement, aggregate and water for each batch shall be determined by weight to an accuracy of 1 part in 1,000.

Mixing Concrete.—The concrete shall be mixed by hand or in a small batch mixer in such a manner as to avoid loss of water. The cement and fine aggregate shall first be mixed dry until the mixture is uniform in colour. The coarse aggregate shall then be added and mixed with the cement and sand. The water shall then be added

and the whole mixed thoroughly for a period of not less than two minutes and until the resulting concrete is uniform in appearance.

Consistence.—The consistence of each batch of concrete shall be measured, immediately after mixing, by the slump test made in accordance with the Method of Test for Consistence of Concrete given in Appendix VI. Providing that care is taken to ensure that no water is lost the material used for the slump tests may be re-mixed with the remainder of the mix before making the test specimen.

Size of Test Cubes.—Compression tests of concrete shall be made on 6-inch cubes. The moulds shall be of steel or cast-iron with inner faces accurately machined in order that opposite sides of the specimens shall be plane and parallel. Each mould shall be provided with a base plate having a plane surface and of such dimensions as to support the mould during filling without leakage and preferably attached by springs or screws to the mould. Before placing the concrete in the mould both the base plate and the mould shall be oiled to prevent sticking of the concrete.

Compacting.—Concrete test cubes shall be moulded by placing the fresh concrete in the mould in three layers, each layer being rammed with a steel bar 15 inches long and having a ramming face of 1 inch square and a weight of 4 lb. For mixes of 1½ inches slump or less, 35 strokes of the bar shall be given for each layer; for mixes of wetter consistence this number may be reduced to 25 strokes per layer.

Curing.—All test cubes shall be placed in moist air of at least 90 per cent. relative humidity and at a temperature of 58° F. to 64° F. for 24 hours (± ½ hour) commencing immediately after moulding is completed. After 24 hours the test cubes shall be marked, removed from the moulds, and placed in water at a temperature of 58° F. to 64° F. until required for test.

Method of Testing.—All compression tests on concrete cubes shall be made between smooth plane steel plates, without end packing, the rate of loading being kept approximately at 2,000 lb. per sq. inch per minute. One compression plate of the machine shall be provided with a ball seating in the form of a portion of a sphere, the centre of which falls at the central point of the face of the plate.

All test cubes shall be placed in the machine in such a manner that the load shall be applied to the sides of the cubes as cast.

Distribution of Specimens and Standard of Acceptance.—For each age at which tests are required, three cubes shall be made and each of these shall be taken from a different batch of concrete.

The acceptance limits are a difference of 15 per cent. of the average strength between the maximum and minimum recorded strengths of the three cubes. In cases where this is exceeded repeat tests shall be made, excepting where the minimum strength test result does not fall below the strength specified.

APPENDIX VIII

STANDARD METHOD OF MAKING WORKS CUBE TESTS OF CONCRETE

The method described applies to compression tests of concrete sampled during the progress of the work.

Size of Test Cubes and Moulds.—The test specimens shall be 6-inch cubes. The moulds shall be of steel or cast iron, with inner faces accurately machined in order that opposite sides of the specimen shall be plane and parallel. Each mould shall be provided with a base plate having a plane surface and of such dimensions as to support the mould during filling without [illegible] and preferably attached by springs or screws to the [illegible] placing the concrete in the mould both the base plate and the mould shall be oiled to prevent sticking of the concrete.

Sampling of Concrete.—Wherever practicable concrete for the test cubes shall be taken immediately after it has been deposited in the work. Where this is impracticable samples shall be taken as the concrete is being delivered at the point of deposit, care being taken to obtain a representative sample. All the concrete for each sample shall be taken from one place. A sufficient number of samples, each large enough to make one test cube, shall be taken at different points so that the test cubes made from them will be representative of the concrete placed in that portion of the structure selected for tests. The location from which each sample is taken shall be noted clearly for future reference.

In securing samples the concrete shall be taken from the mass by a shovel or similar implement and placed in a large pail or other receptacle, for transporting to the place of moulding. Care shall be taken to see that each test cube represents the total mixture of concrete from a given sample. Different samples shall not be mixed together but each sample shall make one cube. The receptacle containing the concrete shall be taken to the place where the cube is to be moulded as quickly as possible and the concrete shall be slightly re-mixed before placing in the mould.

Consistence.—The consistence of each sample of concrete shall be measured, immediately after re-mixing, by the slump test made in accordance with the Method of Test for Consistence of Concrete given in Appendix VI.

Providing that care is taken to ensure that no water is lost the material used for the slump tests may be re-mixed with the remainder of the mix before making the test cube.

Compacting.—Concrete test cubes shall be moulded by placing the fresh concrete in the mould in three layers, each layer being rammed with a steel bar 15 inches long and having a ramming face of 1 inch square and a weight of 4 lb. For mixes of 1½ inches slump or less, 35 strokes of the bar shall be given for each layer; for mixes of wetter consistence the number may be reduced to 25 strokes per layer.

Curing.—The test cubes shall be stored at the site of construction, at a place free from vibration, under damp sacks for 24 hours (± ½

hour) after which time they shall be removed from their moulds, marked and buried in damp sand until the time for sending to the testing laboratory. They shall then be well packed in damp sand or other suitable damp material and sent to the testing laboratory, where they shall be similarly stored until the date of test. Test cubes shall be kept on the site for as long as practicable and for at least three-fourths of the period before test except for tests at ages less than seven days.

The temperature of the place of storage on the site shall not be allowed to fall below 40° F., nor shall the cubes themselves be artificially heated.

Record of Temperatures.—A record of the maximum and minimum day and night temperatures at the place of storage of the cubes shall be kept during the period the cubes remain on the site.

Method of Testing.—All compression tests on concrete cubes shall be made between smooth plane steel plates without end packing, the rate of loading being kept approximately at 2,000 lb. per sq. inch per minute. One compression plate of the machine shall be provided with a ball seating in the form of a portion of a sphere, the centre of which falls at the central point of the face of the plate.

All cubes shall be placed in the machine in such a manner that the load shall be applied to the sides of the cubes as cast.

APPENDIX IX

REQUIREMENTS FOR CLINKER AGGREGATE FOR PRESSED CONCRETE BRICKS AND PRE-CAST BLOCKS TO BE USED IN PANEL WALLS

The clinker aggregate shall consist of well sintered furnace clinker unmixed with any extraneous matter and substantially free from unburnt coal. The context of combustible matter in the aggregate as determined by the loss on ignition of the material after sampling, finely powdering, and drying at 110° C. shall not exceed 8 per cent. The aggregate shall also pass the following soundness test :—

A normal Portland cement conforming to the British Standard Specification and a fine white plaster of Paris (it is most important that no other type of plaster should be used) shall be thoroughly mixed in equal proportions by weight and stored in a dry place in a tin with closely fitting lid until required. This mix shall not be used for the present test when it is older than three weeks.

One part by volume of this cement-plaster mix shall be mixed with five parts of the clinker sample, ground to pass a 76-mesh sieve. The whole shall be thoroughly mixed while dry and any small lumps carefully broken down. It shall be gauged with water to a plastic condition, and then rolled into a lump and placed on the centre of a glass plate, about 4 inches square, which must be clean and not greasy. The mix shall be of such plasticity that it flows outwards towards the edge of the plate when the latter is tapped. The plate shall be tapped until the lump has formed a pat about 3 inches in diameter, which shall be finally shaped by stroking with a knife

from the outside edge towards the centre. The completed pat shall be about ½ inch deep at the centre and taper away to a thin edge at the circumference, as shown in the diagram.

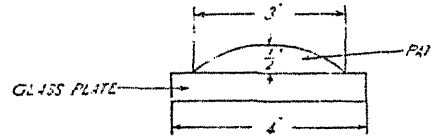


Fig. 481.

Care shall be taken to avoid a tendency of the pat to flatten out excessively owing to making the mix too wet. The gauging and making of the pat shall be completed within five minutes of the time of addition of the water to the mix, or the initial set of the plaster may be destroyed. The pat should set hard within about 15 minutes.

Two pats shall be made, using a fresh dry mix for the second pat, and shall be placed immediately after making in a moist atmosphere, *e.g.*, in a vessel containing water but raised above the water surface, and kept there for at least three hours but not exceeding four hours. The pat shall then be totally immersed in water. Care must be taken not to touch the pat itself, nor to subject it to vibration.

The pats shall be examined at intervals from one to seven days after immersion and if during this period fine radial cracks appear, running from the edge to the centre, or the edges of the pat lift in a marked manner away from the plate, or the pat becomes loose on the plate, the clinker shall be rejected.

L.C.C. Bye-laws.—The bye-laws issued by the London County Council dealing with reinforced concrete construction and in pursuance of the London Building Act (Amendment) Act, 1935, are based upon and very similar to the provisions of the above Code. There are two main differences :—

(*a*) The L.C.C. fix a constant modular ratio, $m = 15$, as against values varying from 11 to 18 in the Code.

(*b*) The L.C.C. define the permissible direct compression stress in longitudinal mild steel in *axially* loaded columns as the calculated compressive stress in the surrounding concrete multiplied by the modular ratio ; in the Code it is taken at a fixed figure of 13,500 lbs./in.^{2}

Bending Moments.—The bending moments for beams supported and fixed at the ends are the same as for ordinary beams and are fully explained in the chapter on Girders. In a ferro frame building, where the girders and pillars are monolithic and where the beams are continued over several spans, they become continuous beams and the value of the

BM's must be computed by the method of the theory of three moments, see chapter on Girders. Here it will be seen that the BM will be considerably reduced in the spans of the girders, but that the loads on some of the pillars will be considerably increased, under any given method of loading. There will also be reverse moments over the supports which materially affect the placing of the steel in the beam.

Exact values for the moments can be obtained by the above method. Where there is any given method of loading that can be relied upon to be static, this method should be employed. In most ordinary cases the loads on floors and on different parts of floors are liable to be variable. The case giving the greatest moment should be calculated. This usually occurs where alternate bays are loaded and unloaded. Although as mathematical exercises the determination of the exact moments is excellent, the values given in the Code, Clause 602, may be employed as these cover the worst conditions. It is advisable in all cases to draw the graph of the BM to ensure the proper disposition of the reinforcements.

Adhesion or Grip.—There is some doubt as to what exactly takes place between the steel and the concretes when used in combinations. Whether the adhesive resistance is caused by a chemical fusion between the surfaces in contact, or to the grip due to the contraction of the concrete in setting. It is probably due to both of these causes. The safe value given for this resistance is 100 lbs. per square inch. The adhesion must be provided for, wherever there is a tendency for slipping to take place, or where joints in the bars are necessary, such as at the points over the supports in continuous beams, or where the turned up bars, that do not run the full length of the beam, are cut off. In such cases the ends of the bars must be continued beyond each other a distance sufficient to counteract the tension in the bar. The length of this overlap is limited by the tensional resistance of the bar and is determined as follows.

Let a round steel bar be embedded in concrete as above and stressed to the maximum of 18,000 lbs. per square inch. Determine the length required such that the adhesion stress

between the steel and concrete shall not exceed 100 lbs. per square inch (see clause 42*a*).

Then Stress in bar = 18000 lbs. per square inch.

$$\text{Cross-sectional area} = \frac{\pi d^2}{4}$$

$$\text{Load carried by bar} = 18000 \times \frac{\pi d^2}{4}$$

$$= 4500\pi d^2$$

$$\text{Area of surface exposed to concrete} = \pi dl$$

Hence $$\frac{4500\pi d^2}{\pi dl} = 100 \text{ lbs. per square inch.}$$

$$4500\, d = 100l$$

$$l = 45d$$

Thus, if the adhesion stress is not to be more than 100 lbs. per square inch the length of bar imbedded in the con-

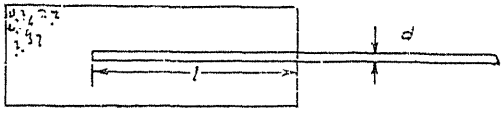


Fig. 482.

crete must not be less than forty-five times the diameter of the bar.

The stress to be provided for in beams is equal to the horizontal shear stress occurring in the particular layer of the concrete surrounding the bar. The intensity of the shear increases as the neutral axis is approached as shown in the intensity diagrams in the article on shear. In practice it is usual to consider the intensity as being uniform throughout the depth of the beam.

Moments of Resistance.—The moment of resistance of any beam is dependent upon the values assumed or evolved for the four factors "*m*" the modular ratio, "*n*" the position of the neutral axis, "*r*" the ratio of the steel to the whole section, and "*a*" the arm of the resistance moment or the lever arm "*m*."

In Fig. 483 and working on the usual assumptions f_c and f_t at equal distances from the NA are equal and the strain $e = \frac{f_c}{E_c} = \frac{f_s}{E_s}$

and $$\frac{E_s}{E_c} = \frac{f_s}{f_c} = m \quad \ldots\ldots\ldots \quad (1)$$

The modulus of elasticity for mild steel $E_s = 30{,}000{,}000$ lbs. and assuming $E_c = 2{,}000{,}000$ lbs. and $\frac{E_s}{E_c} = \frac{30{,}000{,}000}{2{,}000{,}000} = 15 = m$.

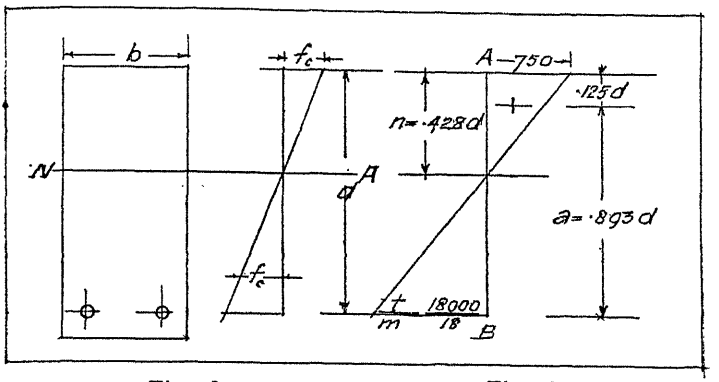


Fig. 483. Fig. 484.

Figs. 483—484.

The value of "m" for concrete varies with the mix. The Code gives a series of values for "m" varying from 11 to 18, according to the mix.

Economic Ratio.—When the ratio of steel to concrete is such that both materials reach their limits of resistance simultaneously, they are said to be in the "economic ratio." The undermentioned values of "r," "n" and "a" are evolved to satisfy this condition. These values can be deduced from the following diagram. To draw the diagram set out a vertical line AB and, on the top right hand, set out a line at right angles 600 units, to any scale. This represents "C" the safe value of concrete. On the bottom left-hand side set out $\frac{t}{m} = \frac{18000}{18} = 1{,}000$ to the same scale. 18,000 is the safe value of steel allowed by the Code, and 18 the value of m for 1 : 2 : 4 concretes. This represents the stress in the concrete at this distance below the NA. Join the extremities of the horizontal, then the

intersection with AB gives the depth below the top surface of the NA (see Fig. 484).

$$n = \left(\frac{c}{\frac{t}{m} + c}\right) d = \left(\frac{750}{1000 + 750}\right) d = 0{\cdot}428d$$

The measurement from the diagram confirms this result.

To determine the ratio of steel r.

$$\begin{aligned}
\text{The compression area} &= \tfrac{1}{2}bn \\
\text{Total compression} &= \tfrac{1}{2}cbn \\
\text{Total tension} &= rbdt \\
\text{Total compression} &= \text{Total tension} \\
\tfrac{1}{2}cbn &= rbdt \\
375 \times b \times {\cdot}428d &= rbd\ 18000 \\
r &= {\cdot}00893.
\end{aligned}$$

The whole of the compression may be assumed to act at the centre of the compression area $\frac{n}{3} = \frac{{\cdot}428d}{3} = {\cdot}143d$ from the top and $a = d - {\cdot}143d = {\cdot}857d$.

Moment of Resistance.—The moment of resistance of a section is the total tension or the total compression multiplied by the lever arm a.

$$\begin{aligned}
\text{Thus} \quad R_t = trbda &= 18000 \times {\cdot}00893 \times b \times d \times {\cdot}857d \\
&= 137{\cdot}6\ bd^2 \\
\text{and} \quad R_c = \tfrac{1}{2}cbna &= \frac{750}{2} \times b \times {\cdot}428d \times {\cdot}857d \\
&= 137{\cdot}6bd^2.
\end{aligned}$$

Thus the general expression for the moment of resistance may be written

$$R = Qbd^2 \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (2)$$

Q being the symbol for the numerical factor 137·6, which is constant for the "economic ratio" for 1 : 2 : 4 concrete.

The following table is a summary of the values of m, n, r, a and Q for the economic ratio for the four standard mixes for the ordinary and high grades of concrete given in the Reinforced Structures Committee Report.

ORDINARY GRADE CONCRETE

	Cement.	Sand.	Ballast.	Cement.	Sand.	Ballast.	Cement.	Sand.	Ballast.	Cement.	Sand.	Ballast.
	1	1	2	1	1·2	2·4	1	1·5	3	1	2	4
m	14			14			16			18		
n	·432d			·418d			·430d			·428d		
r	·1170			·01075			·01015			·00893		
a	·856d			·861d			·857d			·857d		
Q	180·0			166·6			156·6			137·6		

HIGH GRADE CONCRETE

m	11			11			12			14		
n	·433d			·423d			·423d			·425d		
r	·01506			·01414			·01294			·01122		
a	·856d			·859d			·859d			·858d		
Q	232·0			218·8			200·0			173·3		

The values for the factors "n," "r" and "a" can be calculated for beams of rectangular section and for T beams, where the NA falls within or at the bottom surface of the slab.

Notation for Beams and Slabs.

A = *area* of tensile reinforcement, in square inches.

a = *arm* of the resistance moment, in inches.

B = *bending* moment of the external loads and forces.

b = *breadth* of rectangular beam in inches, or *breadth* of the flange of a tee beam in inches or *breadth* of slab in inches.

c = permissible *compressive* working stress, at the extreme edge of the concrete in compression, in pounds per square inch.

d_s = total *depth* of *slab* in inches.

d = effective *depth* of the beam or slab in inches, *i.e.*, the distance from the compressed edge of the constructional concrete to the common centre of gravity of the tensile reinforcement.

E_c = *elastic* modulus of *concrete* in compression.

E_s = *elastic* modulus of *steel* in tension or compression.

l = *length* of the effective span of a beam or slab.

$m = \frac{E_s}{E_c}$ = *modular* ratio.

n = distance of the *neutral* axis from the compressed edge of the constructional concrete of the beam or slab, in inches.

$n_1 = \frac{n}{d}$ = *neutral* axis ratio, $\therefore n_1 d = n$.

p = *percentage* of tensile reinforcement = 100r.

Q = *qualifier* in the equation $R = Qbd^2$.

R = *resistance* moment generally.

R_c = *resistance* moment of the internal stresses in the beam or slab in terms of the permissible *compressive* working stress.

R_t = *resistance* moment of the internal stresses in the beam or slab in terms of the permissible *tensile* working stress.

r = *ratio* of A to bd, *i.e.*, $r = \frac{A}{bd}$ and $A = rbd$.

s_1 = *slab* depth ratio $= \frac{d_s}{d}$

t = permissible *tensile* working stress, in tensile reinforcement, in pounds per square inch.

t_1 = ratio of the *tensile* stress in the steel to the compressive stress at the extreme edge of the concrete under flexure.

$= m\left(\frac{1}{n_1} - 1\right)$.

W = total *weight*, or *working* load.

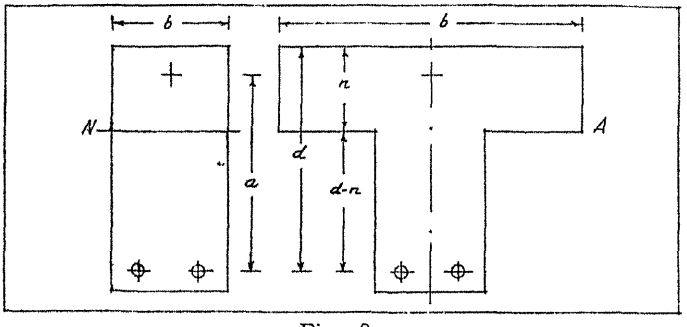


Fig. 485.

To determine the value of "n."

The stress at any section of a beam is proportional to its distance from the neutral axis. In reinforced concrete beams the steel is m times more rigid than the concrete, and

as the concrete is assumed to take none of the tensional stress, at any given distance from the neutral axis, the stress in the steel will be m times that in the concrete.

Thus

$$\frac{m.c}{t} = \frac{n}{(d-n)} = \frac{n_1 d}{(d-n_1 d)}$$

$$\therefore \quad \frac{c}{t} = \frac{n_1}{m(1-n_1)} \qquad \ldots\ldots\ldots (3)$$

The mean stress in the concrete $= \frac{1}{2}c$

The total compressive load to be taken by the concrete $= \frac{1}{2}c \,.\, bn$

The total tensile load to be taken by the steel $= tA = t.rbd$

Equate the total tension and compression

$$\frac{1}{2}c \,.\, bn = trbd$$

$$\frac{c}{t} = \frac{2rd}{n} = \frac{2r}{n_1} \qquad \ldots\ldots\ldots (4)$$

Equate these two values for $\frac{c}{t}$

$$\frac{2r}{n_1} = \frac{n_1}{m(1-n_1)}$$

$$2rm - 2rmn_1 = n_1^2$$

$$n_1^2 + 2rmn_1 - 2rm = 0$$

$$\therefore \quad n_1 = \tfrac{1}{2}\left(-2rm \pm \sqrt{4r^2m^2 + 8rm}\right)$$

$$n_1 = \sqrt{r^2m^2 + 2rm} - rm \qquad \ldots (5)$$

To determine the value of "r."

Thus,

$$tr\left(1 - \frac{n_1}{3}\right) = \frac{cn_1}{2}\left(1 - \frac{n_1}{3}\right)$$

$$\mathrm{tr} = \frac{cn_1}{2}$$

Substitute for n_1 (5),

$$2tr = c\left\{-mr + \sqrt{m^2r^2 + 2mr}\right\}$$

$$= -cmr + c\sqrt{m^2r^2 + 2mr}$$

$$r(2t + cm) = c\sqrt{m^2r^2 + 2mr}$$

Squaring

$$r^2(4t^2 + 4tcm + c^2m^2) = c^2(m^2r^2 + 2mr)$$
$$4t^2r^2 + 4tcmr^2 + c^2m^2r^2 = c^2m^2r^2 + 2mrc^2$$
$$r^2(4t^2 + 4tcm) - 2mrc^2 = 0$$
$$\therefore r = \frac{2mc^2 + \sqrt{4m^2c^4}}{2(4t^2 + 4tcm)}$$
$$r = \frac{mc^2}{2(t^2 + tcm)} \quad . \quad . \quad . \quad . \quad (6)$$

This expression may be written :—

$$r = c \times \frac{mc}{2(t^2 + tcm)}$$

Now if $mc = 18 \times 750 = 13500$

and $t = 18000$ lbs./in.^{2}

$$\therefore r = c \times \frac{13500}{\quad\quad}$$

$$= \frac{c}{84000}$$

Hence,

$$\text{Percentage reinforcement} = \frac{c}{840} \quad . \quad . \quad . \quad . \quad . \quad (7)$$

By substituting the particular value for c in the above one obtains the exact percentage reinforcement at the economic ratio.

To find the value of r such that the neutral axis is at the underside of the slab.

$$d_s = n$$

Now $n = n_1 d$ and $d_s = s_1 d$

$\therefore$ since $d_s = n$

$$s_1 = n_1 = \frac{s_1^2 + 2mr}{2(s_1 + mr)} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (8)$$

$$2s_1^2 + 2mrs_1 = s_1^2 + 2mr$$
$$2mr(1 - s_1) = s_1^2$$
$$\therefore r = \frac{s_1^2}{2m(1 - s_1)} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (9)$$

Lever arm "a." From the diagram (Fig. 485).

Then the lever arm

$$a = \frac{2}{3}n + (d - n)$$
$$= \frac{2n_1 d}{3} + d - n_1 d$$

$$3a = 2n_1 d + 3d - 3n_1 d$$
$$= 3d - n_1 d$$
$$a = d - \frac{n_1 d}{3}$$
$$\text{lever arm} = d\left(1 - \frac{n_1}{3}\right) \quad \ldots\ldots\ldots \quad (10$$

Tee Beams.—In ferro floors a portion of the floor slab may be included as part of the beam, forming a compressional flange. The amount of the slab that may be included in the flange is indeterminate, but arbitrary values are given for the widths.

Consider the case of a Tee beam in which the neutral axis is outside the slab. Proceed in the same manner as before to obtain values for n_1 and the compressive and tensile resistance moments.

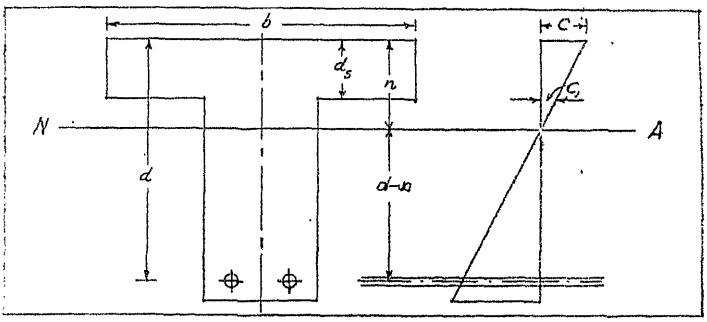


Fig. 486.

Considering the stress at any point to be proportional to its distance from the neutral axis, taking into account the rigidity of the steel, it has been shown that, from (3)

$$\frac{c}{t} = \frac{n_1}{m(1 - n_1)}$$

The amount of compression in the beam between the under side of the slab and the neutral axis is neglected. Then the mean compressive stress in the concrete is the mean between the stress c on the outsize edge and the stress at the under side of the slab

Consider the stress triangle above

$$\frac{\text{Stress at under edge}}{c} = \frac{n - d_s}{n}$$

$$\therefore \quad \text{Stress at under edge} = c \cdot \frac{n - d_s}{n} \quad . \quad . \quad . \quad . \quad (11)$$

Hence the mean compressive stress in the flange

$$\frac{1}{2}\left\{c + c \cdot \frac{n - d_s}{n}\right\} = \frac{c}{2} \cdot \frac{2n - d_s}{n}$$

$$n = n_1 d$$

$$d_s = s_1 d$$

Above expression becomes :

$$\frac{c}{2} \cdot \frac{2n_1 d - s_1 d}{n_1 d} = c\left(1 - \frac{s_1}{2n_1}\right)$$

Then total compressive load $= cbd_s\left(1 - \frac{s_1}{2n_1}\right)$

$$= cbs_1 d\left(1 - \frac{s_1}{2n_1}\right) \quad . \quad . \quad . \quad . \quad . \quad (12)$$

Again total tensile load $= trbd$. . . . . (13)

Equate the total tension and total compression

$$trbd = cbs_1 d\left(1 - \frac{s_1}{2n_1}\right)$$

$$\frac{c}{t} = \frac{r}{s_1\left(1 - \frac{s_1}{2n_1}\right)} = \frac{2rn_1}{s_1(2n_1 - s_1)}$$

Equate now the two values we have obtained for $\frac{c}{t}$

$$\frac{n_1}{m(1 - n_1)} = \frac{2rn_1}{s_1(2n_1 - s_1)}$$

$$2rm(1 - n_1) = s_1(2n_1 - s_1)$$

$$2rm - 2rmn_1 = 2n_1 s_1 - s_1^2$$

$$2n_1(s_1 + mr) = s_1^2 + 2rm$$

$$\therefore \quad n_1 = \frac{s_1^2 + 2mr}{2(s_1 + mr)} \quad . \quad . \quad . \quad . \quad . \quad . \quad (14)$$

To find the lever arm. Consider the section of stress diagram ABCD (Fig. 487). It is required to find the distance of its centre of pressure from AB.

The centre of pressure of the triangle passes through F, such that $EF = \frac{d_s}{3}$. The centre of pressure of the rectangle passes through EH such that $EH = \frac{d_s}{2}$.

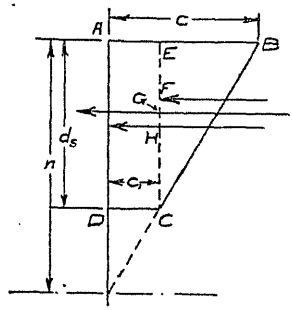


Fig. 487.

$$\text{Area of triangle} = \frac{d_s(c - c_1)}{2}$$

$$\text{Area of rectangle} = d_s c_1$$

$$\text{Total area} = \frac{d_s(c + c_1)}{2}$$

Now $$FH = \frac{d_s}{2} - \frac{d_s}{3} = \frac{d_s}{6}$$

Hence $$FG = \frac{d_s}{6} \times \frac{2d_s c_1}{d_s(c + c_1)}$$

$$= \frac{d_s}{3}\left\{\frac{c_1}{c + c_1}\right\}$$

$$\therefore \quad EG = \frac{d_s}{3} + \frac{d_s}{3}\left\{\frac{c_1}{c + c_1}\right\}$$

$$= \frac{d_s}{3}\left\{\frac{2c_1 + c}{c + c_1}\right\}$$

This equation gives the depth of the centre of pressure from AB.

Substitute the value for c_1 (equation 11) found above.

$$\therefore \quad EG = \frac{d_s}{3}\left\{\frac{\frac{2c(n-d_s)}{n}+c}{c+\frac{c(n-d_s)}{n}}\right\}$$

$$= \frac{d_s}{3}\left\{\frac{2c(n-d_s)+cn}{cn+c(n-d_s)}\right\}$$

$$= \frac{d_s}{3}\left\{\frac{3cn-2cd_s}{2cn-cd_s}\right\}$$

$$= \frac{d_s}{3}\left\{\frac{3n-2d_s}{2n-d_s}\right\}$$

The lever arm $= GK$

$$= AK - EG$$

$$= d - \frac{d_s}{3}\left\{\frac{3n-2d_s}{2n-d_s}\right\}$$

$$= \frac{3d(2n-d_s)-d_s(3n-2d_s)}{3(2n-d_s)}$$

$$= \frac{6nd-3d_sd-3nd_s+2d_s^2}{3(2n-d_s)}$$

Substitute $d_s = s_1 d$

$n = n_1 d$

Lever arm $= \frac{6n_1d^2-3s_1d^2-3n_1s_1d^2+2s_1^2d^2}{3(2n_1-s_1)d}$

$$= \frac{d(6n_1-3s_1(1+n_1)+2s_1^2)}{3(2n_1-s_1)} \quad \ldots\ldots (15)$$

or $$= d\left\{1-\frac{s_1}{3}\left(\frac{3n_1-2s_1}{2n_1-s_1}\right)\right\} \quad \ldots\ldots (16)$$

The resistance moment in terms of the compressive stress is the lever arm multiplied by the compressive load.

To determine the moment of resistance R_c in terms of the compressive stress.

From (12), compressive load $= cbd_s\left(1-\frac{s_1}{2n_1}\right)$

Hence

$$R_c = cbd_s\left(1-\frac{s_1}{2n_1}\right)\times\frac{d(6n_1-3s_1(1+n_1)+2s_1^2)}{3(2n_1-s_1)}$$

$$= \frac{cbdd_s(2n_1-s_1)}{2n_1}\times\frac{6n_1-3s_1(1+n_1)+2s_1^2}{3(2n_1-s_1)}$$

$$= \frac{cbdd_s}{6}\cdot\left\{\frac{6n_1-3s_1(1+n_1)+2s_1^2}{n_1}\right\}$$

Substitute for n_1 as in formula (14).

$$R_c = \frac{cbdd_s}{6}\left\{\frac{6\,.\,(s_1^2 + 2mr)}{2(s_1 + mr)} - 3s_1\left(1 + \frac{s_1^2 + 2mr}{2(s_1 + mr)}\right) + 2s_1^2\right\} \Big/ \frac{s_1^2 + 2mr}{2(s_1 + mr)}$$

$$= \frac{cbdd_s}{6}\left\{\frac{6s_1^2 + 12mr - 3s_1\,.\,2(s_1 + mr) - 3s_1\,.\,(s_1^2 + 2mr) + 2s_1^2(2s_1 + 2mr)}{2(s_1 + mr)} \Big/ \frac{s_1^2 + 2mr}{(2s_1 + mr)}\right\}$$

$$= \frac{cbdd_s}{6}\,.\,\left\{\frac{6s_1 + 12mr - 6s_1^2 - 6s_1mr - 3s_1^3 - 6s_1mr + 4s_1^3 + 4s_1^2mr}{s_1^2 + 2mr}\right\}$$

$$R_c = cbdd_s\left\{\frac{s_1^3 + 4mrs_1^2 - 12mrs_1 + 12mr}{6(s_1^2 + 2mr)}\right\} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (17)$$

Similarly, the resistance moment in terms of the tensile stress.

$$\text{Total tension} = trbd$$

$$\therefore \qquad R_T = \frac{d\{6n_1 - 3s_1(1 + n_1) + 2s_1^2\}}{3(2n_1 - s_1)} \times trdb$$

$$= trbd^2\frac{\{6n_1 - 3s_1(1 + n_1) + 2s_1^2\}}{3(2n_1 - s_1)}$$

Substitute for n_1 as before.

$$R_T = trbd^2\left\{\frac{6\,.\,s_1^2 + 2mr}{2(s_1 + mr)} - 3s_1\left(1 + \frac{s_1^2 + 2mr}{2(s_1 + mr)}\right) + 2s_1^2 \Big/ 3\left\{\frac{2\,.\,(s_1^2 + 2mr)}{2(s_1 + mr)} - s_1\right\}\right\}$$

$$= trbd^2\left\{\frac{6s_1^2 + 12mr - 6s_1^2 + 6s_1mr - 3s_1^2 - 6s_1mr + 4s_1^2(s_1 + mr)}{2(s_1 + mr)} \Big/ \frac{6s_1^2 + 12mr - 6s_1(s_1 + mr)}{2(s_1 + mr)}\right\}$$

$$= trbd^2\left\{\frac{6s_1^2 + 12mr - 6s_1^2 - 6s_1mr - 3s_1^3 - 6s_1mr + 4s_1^3 + 4s_1^2mr}{12mr - 6s_1mr}\right\}$$

$$R_T = tbd^2\,.\,\frac{(s_1^3 + 4mrs_1^2 - 12s_1mr + 12mr)}{6m(2 - s_1)} \quad . \quad . \quad . \quad (18)$$

To find the ratio of reinforcements to give the maximum stresses in the concrete and steel, proceed as before and equate R_T with R_c.

$$R_c = R_t$$

The section of the numerator in the bracket cancels and we have left

$$\frac{cd_s}{s_1^2 + 2mr} = \frac{td}{m(2 - s_1)}$$

$$cd_s m(2 - s_1) = td(s_1^2 + 2mr)$$

$$2cd_s m - cd_s m s_1 = tds_1^2 + 2mrtd$$

Substitute $d_s = s_1 d$

$$2cs_1 m - cs_1^2 m = ts_1^2 + 2mrt$$

$$\therefore \quad r = \frac{2cs_1 m - cs_1^2 m - ts_1^2}{2mt} \quad . \quad . \quad (19)$$

The following table gives a summary of the formulæ required to calculate rectangular and T section beams.

	T BEAMS.	RECTANGULAR SECTIONS.
m	According to mix.	
n_1	$\frac{s_1^2 + 2mr}{2(s_1 + mr)}$	$\sqrt{m^2r^2 + 2mr} - mr$
r	$\frac{2cs_1 m - cs_1 m - ts_1^2}{2mt}$	$\frac{mc^2}{2(l^2 + lcm)}$
r for $n_1 = s_1$	$\frac{s_1^2}{2m(1 - s_1)}$	. . . . .
a	$d\left\{1 - \frac{s_1}{3}\left(\frac{3n_1 - 2s_1}{2n_1 - s_1}\right)\right.$	$d\left(1 - \frac{n_1}{3}\right)$
At	rbd	rbd
Rt	$tbd^2\left(\frac{s_1^3 + 4mrs_1^2 - 12mrs_1 + 12mr}{6m(2 - s_1)}\right)$	$trbd^2\left(1 - \frac{n_1}{3}\right)$ or Qbd^2
Rc	$cbdd_s\left\{\frac{s^3 + 4mrs_1^2 - 12mrs_1 + 12mr}{6(s_1^2 + 2mr)}\right\}$	$\frac{cbd^2n_1}{2}\left(1 - \frac{n_1}{3}\right)$ or Qbd^2

Compressional Reinforcement.—In cases where head-room is limited or where for other considerations the depth of the beam has to be reduced, it may be impossible to employ the "economic ratio." A larger ratio of tensional steel must be employed and the concrete in compression thus becomes overstressed. To avoid this, steel may be used in conjunction with the concrete in the compressional area. Under these conditions advantage cannot be taken of the full resistance of the steel; it will be reduced to that of the surrounding concrete at the particular distance from the NA at which the steel is placed.

The breadth and depths of beams is arbitrary and having fixed these, the usual procedure is to assume the

conditions as for a beam designed for the economic ratio. Thus the values of "n" and "a" are known. The B is then computed. The tensional steel in obtained from formula.

$$A_t = \frac{B}{ta} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (20)$$

Then the total tension = total compression.

$$A_t t = \tfrac{1}{2}\, cbn + A_c(m - 1)\tfrac{2}{3}c$$

and

$$A_c = \frac{tA_t - \frac{1}{2}cbn}{\frac{2}{3}c(m - 1)} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (21)$$

In T beams where $d_s < n$

$$A_c = \frac{tA_t - \frac{1}{2}cbd_s}{\frac{2}{3}c(m - 1)} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (22)$$

In cases where A_c exceeds A_t, it is better to ignore the concrete in compression and stress the steel to its full value.

Slabs.—Slabs are wide, shallow, rectangular beams; they are calculated as such, employing the "economic ratio" for proportioning the steel and concrete. Slabs may be either supported or fixed at their edges, or continuous over several spans. The bending moments would be as for beams under similar conditions of end support. A reduction in the assumed loads may be made, in cases where the length of the slab is less than twice the width, due to the fact that a certain proportion of the weight is considered as being carried in the direction of the length of the slab, and the remainder as being carried in the direction of the width of the slab. The bending moment is then calculated from the usual formulæ, considering the slab lengthways as a beam, and then the bending moment is calculated across the slab, again considering it as a beam.

The reduction formulæ given in this section have been derived from consideration of the fact that the deflection at any point is the same, whether that point be considered as situated on a cross section parallel to the length or a cross section parallel to the breadth.

It must be noted that the curvature will naturally be greater across the slab than along it. Hence the bending

moment will be greater on a transverse cross section, since the bending moment is proportional to the curvature.

Clause 602 of the Code gives a graph and table of coefficients of reduction for values of $\frac{\text{breadth}}{\text{length}}$ from 1 — 3 by which the BM calculated as for an ordinary beam can be reduced.

Example.—Determine the thickness and the reinforcement for a slab continuous over several supports, to carry a superload of 100 lbs. per square foot. The slab is 12′ 0″ × 8′ 0″. The finishings of the floor to consist of ½-inch plaster ceiling, 2-inch weak concrete to contain conduits, etc., above the structural slab, and a 1¼-inch wood block floor set in mastic.

There are two loads to be considered, the dead load, which is constant and uniformly distributed, and the superload which may be concentrated or uniformly distributed. In slabs it is usual to consider the latter and to combine it with the dead load.

The depth of the required slab, being unknown, must be assumed in order to compute its weight. If the rule of allowing ½ inch thickness for every 1 foot in span be taken, then the weight of the slab may be approximated without any sensible error.

Then the load per super foot would be estimated as follows :—

Slab at 4″ × 1′ × 1′ × 144 lbs.	=	48 lbs.
Ceiling at ½″ × 1′ × 1′ × 12 lbs.	=	6 ,,
Surface concrete at 2″ × 1′ × 1′ × 144 lbs.	=	24 ,,
1¼″ wood floor in mastic	=	4 ,,
Superload	=	100 ,,
	Total	182 lbs.

Then taking a strip 12 inches wide the total load W will be

$$W = 182 \times 8 = 1456 \text{ lbs.}$$

and from the tables, Clause 602 of the Code,

$$M_x = z_x \frac{Wl}{8}$$

$$= \frac{\cdot 835 \times 1456 \times 96}{8} = 14600 \text{ lb. ins.}$$

$$M = Qbd^2 = 137{\cdot}6 \times 12 \times d^2$$

$$d = \sqrt{\frac{14600}{137{\cdot}6 \times 12}} = 3 \text{ inches, say.}$$

Use $\frac{3}{8}$" dia. rods, area = 0·1104 square inches.

$$A_t = rbd \text{ and } b = \frac{A_t}{rd}$$

and (b) will be equal the width of concrete for one rod,

$$b = \frac{0{\cdot}1104}{0{\cdot}00893 \times 3}$$

$$= 4{\cdot}12 \text{ inches, say, 4-inch pitch.}$$

Distributing bars are required at right angles to the above bars at distances not exceeding four times the effective depth of the slab, say at 12-inch centres.

The cover required for the rods is $\frac{1}{2}$ inch; this with the half diameter of the rod equals $\frac{11}{16}$ inch. The total depth of the slab would then be $3\frac{11}{16}$ inches, say, 4 inches.

As the slabs are continuous over the supports, there will be a reverse moment at that part. The rods will therefore have to be bent up over the supports. The bends should be approximately over the points of contraflexure which will be about $\frac{l}{5}$ from the supports. The ends of the rods should be hooked and carried beyond the supports to obtain the necessary bond, a distance equal to about forty times the diameter of rod, = 15 inches if straight, or twenty times the diameter, = 9 inches if hooked at ends. Provision for shearing is rarely made in slabs, as there is usually an excess of strength in this direction.

Rectangular Sections.—These usually occur as lintels or bressummers to support a wall over an opening. They may be calculated to comply with the requirements of the "economic ratio" where the conditions as to the depth are not limited. In most cases the breadth would be fixed by the thickness of the wall that it supports, and the depth is generally determined by other considerations.

EXAMPLE.—Determine the longitudinal reinforcements for a ferro beam, supported at ends, to carry a $1\frac{1}{2}$ brick wall over a 20 feet span. The abutments on each side of the opening are ample and the load may be taken as the weight of the mass of brickwork enclosed within an equi-

lateral triangle. The weight of the brickwork to be 112 lbs. per cubic foot. The stresses on concrete and steel to be 600 and 18,000 lbs. per square inch respectively.

Then the load W will equal

$$W = \tfrac{1}{2}\left(20 \times \frac{1{\cdot}73 \times 20}{2}\right) \times \frac{13\tfrac{1}{2}''}{12} \times 112$$

$$= 21800 \text{ lbs.}$$

Then $$M = \frac{Wl}{6} = \frac{21800 \times 240}{6}$$

$$= 872000 \text{ lbs. ins.}$$

$$M = Qbd^2 \text{ and } d = \sqrt{\frac{M}{Q \times b}}$$

$\therefore$ $$d = \sqrt{\frac{872000}{137{\cdot}6 \times 12}} = 23 \text{ inches}$$

$$A_t = rbd = 0{\cdot}00893 \times 13{\cdot}5 \times 23$$

$$A_t = 2{\cdot}77 \text{ inches.}$$

If it is desired to reduce the depth of the beam in the preceding example, it could be done by employing reinforcement in the compressive area as follows. Let the required effective depth be 12 inches.

$$a = 0{\cdot}857 \times 12 = 10{\cdot}3 \text{ inches and}$$

$$n = 0{\cdot}428 \times 12 = 5{\cdot}14$$

Then $$A_t = \frac{M}{ta} = \frac{872000}{18000 \times 10{\cdot}3}$$

$$= 4{\cdot}7 \text{ square inches.}$$

Then from formula (21) for compressive reinforcement

$$A_c = \frac{tA_t - \tfrac{1}{2}cbn}{\tfrac{1}{2}c(m-1)}$$

$$= \frac{(18000 \times 4{\cdot}7) - (375 \times 13{\cdot}5 \times 5{\cdot}14)}{500\,(18-1)}$$

$$= 6{\cdot}9 \text{ square inches.}$$

This is much greater than the tensile reinforcement due to the steel only, having a resistance of mc, *i.e.*, $18 \times 500 = 9{,}000$ lbs. per square inch instead of 18,000 lbs. per square inch, to which it could be stressed if it acted independently of the concrete. Five hundred pounds per square inch is the stress in the concrete at two-thirds the distance from the NA. By ignoring the resistance of the

concrete the compressional reinforcement will be equal to the tensile, *i.e.*, 4·7 square inches. Then if four 1¼-inch bars are placed in the centres of tension and compression respectively, there would be a slight excess of strength.

Allowing 2 inches protection at the bottom, this would give an overall depth of 14 inches, being a reduction of 11 inches over the preceding example. In cases where the depth is immaterial, the question of cost would decide which method to adopt.

Wherever compressional reinforcement is employed great care must be taken that the compressional steel is securely anchored, the binding steel used for this purpose being placed at a maximum pitch of sixteen times the diameter of the bars where the resistance of the concrete is considered and eight times the diameter of the bars where the steel is designed to take the whole of the compression. This is to prevent the compressional bars buckling when fully stressed.

Tee Beams.—In the paragraph dealing with slabs, it was shown that the depth of the slab varied with the span, and that if the span was large the depth would be very great, but that as all the concrete below the NA was ignored for stress-resisting purposes, it could be omitted

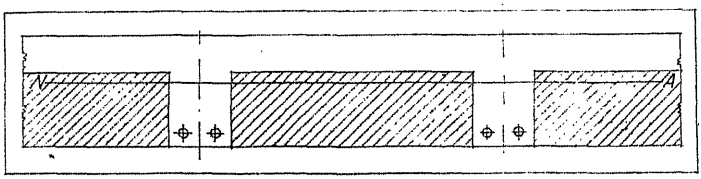


Fig. 488.

by concentrating the tensional steel at points and surrounding it with sufficient concrete only for protection. Thus this arrangement gives a thin slab with a series of projecting ribs on the under surface. The hatched portion in Fig. 488 shows the amount of concrete dispensed with. A portion of the slab directly above the ribs constitutes the compression flange of a girder, of which the longitudinal steel forms the tension flange.

The formulæ that have been given are not adapted

for determining directly the dimensions and proportions of a beam. These must be assumed, and the desired result arrived at by trial. These formulæ are very cumbersome in operation, and more direct if approximate methods are employed. Any slight errors that occur by the use of these methods are negligible.

The following example demonstrates the direct method of computation by the approximate method.

EXAMPLE.—Determine the reinforcements for the secondary beams, 20 feet span, supporting a slab at 8 ft. 4 in. centres, carrying a superload of 100 lbs. per foot super.

First determine the slab, the load being made up as follows. Assume a slab 5 inches thick

W of slab $= \frac{5}{12} \times \cdot 144 = 60$ lbs. per ft. super.

,, finishings $= 35$

,, superload $= 100$

195 lbs. ft. sup.

Say, 200 lbs.

Take a strip 12 inches wide

Total load W $= 8 \cdot 3 \times 200 = 1660$ lbs.

Refer to table, p. 692.

$$M_x = z_x \frac{wl_x^2}{8} = z_x \frac{wl_x}{8}$$

$$\frac{l_y}{l_x} = \frac{20'-0''}{8'-4''} = 2\tfrac{1}{2} \text{ approx.}$$

$\therefore \quad z_x = 0 \cdot 975$

And $\quad M_x = 0 \cdot 975 \cdot \frac{1600 \times 100}{8}$

$= 20200$ lbs. inches.

$$M = Qbd^2$$

$$\therefore \quad d = \sqrt{\frac{M}{Qb}} = \sqrt{\frac{20200}{137 \cdot 6 \times 12}}$$

$= 3 \cdot 5$ inches.

Add $\frac{3}{4}$ inch for cover, then total depth $= 4\frac{1}{4}$ inches.

Assume $\frac{1}{2}$ inch dia. rods; area 0·1963 square inches.

$$A_t = rbd.$$

Consider b the width served by one rod or pitch

Then $\quad b = \frac{0 \cdot 1963}{0 \cdot 00893 \times 3 \cdot 5} = 6 \cdot 28$ inches

say 6-inch pitch

Maximum pitch of distributing bars must not exceed four times the effective depth of the slab (see Code, para. 504).

$$4 \times 3{\cdot}5'' = 1'-2'' \text{ centres.}$$

Tee Beam.—Assume a rib 16 inches effective depth by 10 inches in breadth. Assume depth of rib below slab at 14 inches.

Weight of slab $= 8{\cdot}3 \times 20 \times 200 \quad = 33200$

Weight of rib $= 20 \times \frac{14}{12} \times \frac{10}{12} \times 144 = 2800$

36000 lbs.

$$M = \frac{Wl}{10} = \frac{36000 \times 240}{10}$$

$$= 864000 \text{ in. lbs.}$$

Value of a is assumed and is measured from the centre of slab.

$$A_t = \frac{M}{ta} = \frac{864000}{18000 \times 13{\cdot}88}$$

$$= 3{\cdot}45 \text{ square ins.}$$

Select No. 4/1⅛ inch dia. bars. A = 3·98 square ins.

Assume the average resistance c of the compressional flange at $\frac{750}{2} = 375$ lbs. per square inch.

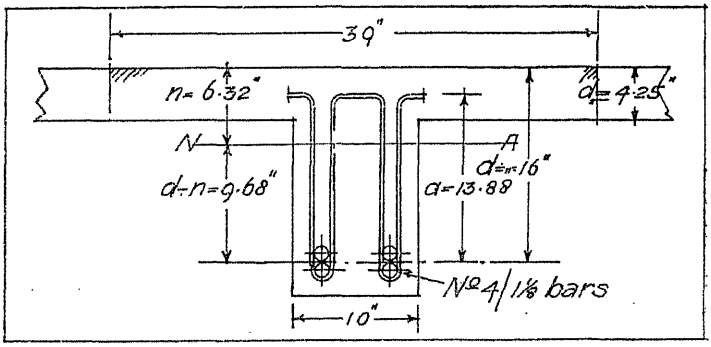


Fig. 489.

Then $M = A_c ca$

$$A_c = \frac{M}{ca} = \frac{864000}{375 \times 13{\cdot}88}$$

$$= 166 \text{ square ins.}$$

$$\text{Width of flange} = \frac{\text{Area of flange}}{\text{Thickness of flange}} = \frac{166}{4{\cdot}25} = 39 \text{ inches.}$$

This width is ample to comply with conditions in Section 6 of Code, clause (*g*). To check if the assumed resistance of the flange is within the limits of safety, it will be necessary to find r and n for the section.

$$r = \frac{A_t}{bd} = \frac{3{\cdot}98}{39 \times 16} = 0{\cdot}00638$$

To determine n take moments about the top edge of the slab. Neglect the concrete between the underside of the slab and the neutral axis.

The equivalent area of concrete on the tensile side of the neutral axis is obtained by multiplying the area of steel by m.

Then $A = (39 \times 4{\cdot}25) + (18 \times 3{\cdot}98) = 237$ square ins.

Taking moments

$$A\,n = bd_s \cdot \frac{d_s}{2} + m\,A_t\,d$$

$$237\,n = \frac{39 \times 4{\cdot}25 \times 4{\cdot}25}{2} + 18 \times 3{\cdot}98 \times 16$$

$$n = 6{\cdot}32 \text{ inches.}$$

Then the average resistance per square inch along the centre line of the compression flange equals

$$\frac{750 \times 4{\cdot}2}{6{\cdot}32} = 497 \text{ lbs. per square inch.}$$

so that the assumption of 375 lbs./square inch was well on the side of safety.

Shearing.—The general principles of shearing stresses have been explained in the chapter on girders to which the reader is referred. It is there demonstrated that the action of the vertical and horizontal shearing couples is to cause tensional and compressional diagonal stresses.

For this reason ferro beams tend to fracture along lines approximately at right angles to the lines of diagonal tension. Where the area of the concrete is insufficient to resist the tensional stress, steel is introduced to take the whole of the tension.

The steel members added are arranged in combination with the concrete to form in effect a series of N or lattice girders, according to whether the steel is placed upright

Fig. 490.

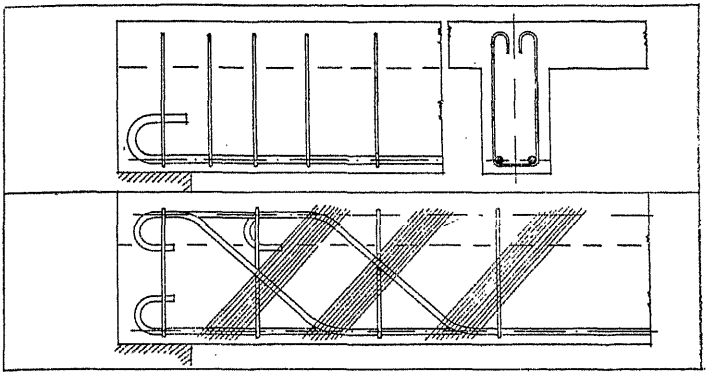


Fig. 491.

Figs. 490—491.

or inclined (see Figs. 490 and 491), either in the form of stirrups or some of the longitudinal reinforcements are turned up and anchored in the compressional area.

Stirrups.—In steel-trussed girders of the type mentioned the bays of the lattice are made uniform in size, and the section of the bars are varied according to the intensity of the stress. In ferro work it would be inconvenient to vary the section of the bars. The area of these, therefore, is kept uniform, but the pitch is varied.

Where inclined bars are used, their number is always limited. They may or may not be sufficient to take up the whole of the tension. If inadequate, stirrups are employed to make up the deficiency; but in any case

stirrups are employed, being placed at a maximum pitch equal to the lever arm a. This ensures that in every case a stirrup will traverse all tensional planes.

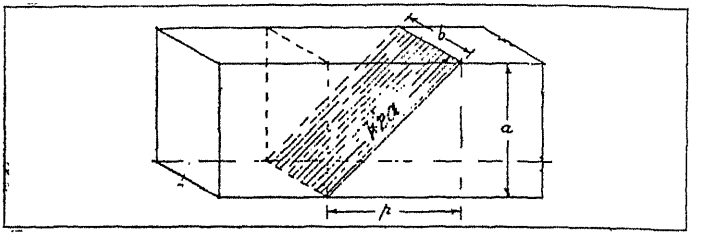


Fig. 492.

From Fig. 492 it will be seen that the area of the diagonal plane

$$A = \sqrt{2}.ab$$

and the tension

$$T = \text{area} \times \text{stress}$$
$$= \sqrt{2}.abt$$

The vertical and horizontal components of this

$$T_h \text{ and } T_v = \frac{T}{\sqrt{2}} = \frac{\sqrt{2}.abt}{\sqrt{2}}$$
$$T_h = T_v = abt.$$

The safe allowable shear stress on concrete is 75 lbs.

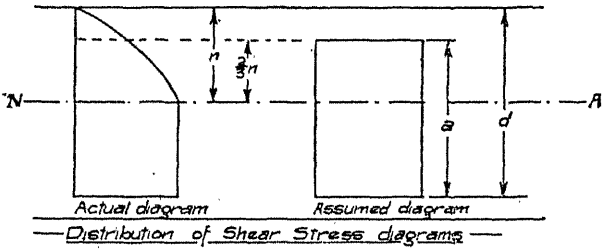


Fig. 493.

per square inch. Therefore, if S_v exceeds $75 \times ab$ then steel reinforcements must be provided. If advantage is to be taken of the full strength of the steel, the resistance of the concrete must be ignored and steel sufficient to take the whole of the stress must be employed. The stirrups must be taken round the longitudinal steel and at least to the upper limit of a and hooked to anchor them properly and to prevent slipping (see Fig. 490).

Fig. 493 shows the actual shear distribution diagram, and the distribution assumed for purposes of calculation. The shear is assumed to be uniformly distributed between the upper and lower limits of a.

The relation between the area of the steel required and the stress can be seen from the following:

$$\text{Stress} = \frac{\text{total shear}}{\text{area}} = \frac{S_h}{bp} = \frac{S_v}{ab}$$

$$\therefore \quad S_h = \frac{S_v p}{a} \quad . \quad . \quad . \quad . \quad (23)$$

but $$S_h = A_s s = \frac{S_v p}{a}$$

$$\therefore \quad p = \frac{A_s s a}{S v}$$

Spacing of Stirrups.—In most cases it is found that the shearing forces are greatest at the ends of the beam, *i.e.*, near the supports. It is necessary then to vary the provision that is made for resisting the shear throughout the length of the beam. It is not usual to vary the size of the stirrups, so the value of p, or the spacing, is varied. The correct spacing for the stirrups is best done by a graphical method shown below.

It has been shown before that the ordinate of a BM diagram at any distance along the beam is equal to the area of the shear force diagram up to that point.

Thus in the case shown in Fig. 494 the maximum bending moment ordinate is equal to the area of the shear force diagram from support to centre.

$$\text{Max. BM} = \frac{wl}{2} \times \frac{l}{2} \times \frac{1}{2} = \frac{wl^2}{8}$$

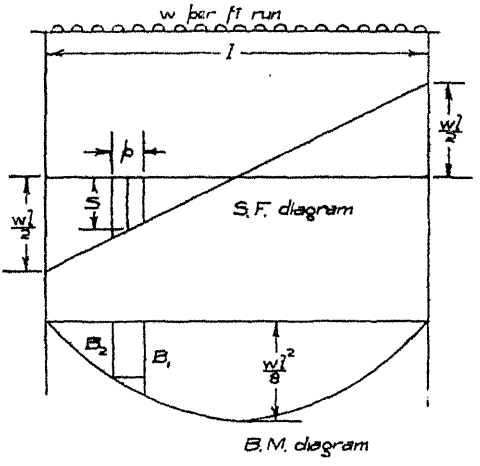


Fig. 494.

From this it can be seen that the difference between two ordinates B_1 and B_2 will be equal to the area of the strip of shear force diagram above it. Hence

$$Sp = B_1 - B_2$$

$$\therefore \quad S = \frac{B_1 - B_2}{p}$$

Considering p as being the distance between two stirrups we have

$$S_v = \frac{B_1 - B_2}{p}$$

Now $$S_h = \frac{S_v p}{a} \text{ from (23)}$$

Substituting $$S_h = \frac{B_1 - B_2}{a} \quad \ldots\ldots\ldots \quad (24)$$

S_h is the quantity with which we are concerned in calculating the area of the stirrups. Since the stirrups are to be alike S_h must be constant. Hence $\frac{B_1 - B_2}{a}$ must be constant. Since a is a constant

$\therefore$ $B_1 - B_2$ must be constant.

The graphical method for determining the spacing of the stirrups is based upon the above fact. The following diagram will indicate clearly the method.

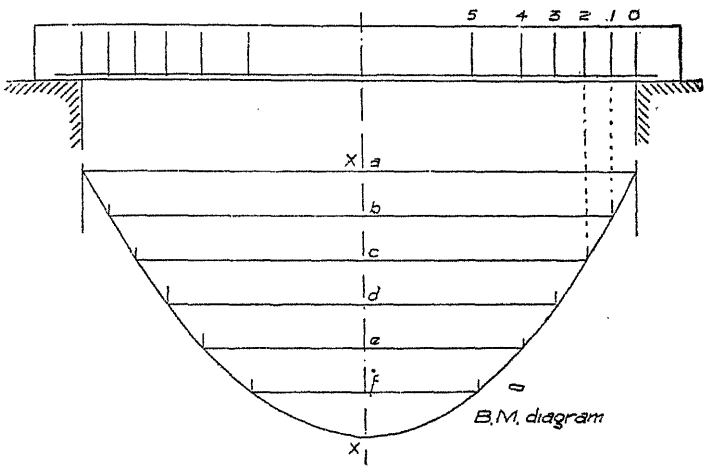


Fig. 495.

The bending moment diagram for the particular system of loading is drawn. The maximum ordinate XX is drawn and divided into a number of equal parts. Through each of these points is drawn a line parallel to the closing line of the diagram. Where these lines cut the curve verticals are drawn which give the positions of the vertical shear reinforcement. Since the vertical distances *ab*, *bc*, *cd*, etc., the difference in bending moment between the positions 1–2, 2–3, 3–4, etc., is constant. This is what was required to be done, *i.e.*, $B_1 - B_2$ is a constant.

In order to determine the number of divisions into which XX must be divided it is necessary to consider the first vertical reinforcement. From (23) the pitch of the stirrups, 0–1 (Fig. 495), $p = \frac{A_s s a}{S_v}$. Set this distance horizontally on the beam and project it downward on to the curve. This gives the depth of the layer *ab*, *bc*, etc. From

the point where each successive layer cuts the curve project up on to the beam. This gives the correct position of each stirrup.

It frequently happens that no reinforcement in shear is required, there being sufficient area of concrete to safely resist the force, but it must be clearly understood that the shear force must be wholly resisted by the concrete or wholly by the steel. The code allows for 1 : 2 : 4 concrete, 75 lbs. per square inch. Stirrups will not be required to resist shear if the section is not stressed in excess of that amount.

That is $$s = \frac{S}{ab}$$

where s is the stress per square inch.

EXAMPLE, *Case* (1) (p. 742).—The shear at the supports.

$$S = \frac{W}{2} = \frac{21800}{2} = 10900 \text{ lbs.}$$

$$a = 0{\cdot}857 \times 23 = 19{\cdot}7 \text{ inches.}$$

$$s = \frac{S}{ab} = \frac{10900}{19{\cdot}7 \times 13{\cdot}5}$$

$$= 41 \text{ lbs. per sq. inch.}$$

Thus no reinforcement is required, but it is usual to place stirrups spaced at a distance not exceeding the "arm of the resistance moment a."

EXAMPLE, *Case* (2) (p. 742).—In the second example, where the depth is reduced to 15 inches, S = 10,900 lbs.

$$s = \frac{S}{ab} = \frac{10900}{10{\cdot}3 \times 13{\cdot}5}$$

$$= 78{\cdot}3 \text{ lbs. per square inch.}$$

This stress would usually be regarded as near enough to the safe stress of 75 lbs./in.^{2} Stirrups, however, are required to anchor the compressive reinforcement and prevent the bars from buckling. The Code provides that they shall be placed not more than eight times the diameter of the bars apart. Thus the pitch of the stirrups would be $8 \times 1\frac{1}{2}'' = 9$ inches.

Inclined Bars.—In girders under a distributed load and in many other cases, the whole of the bars in the longitudinal reinforcement are not required to run through the full length of the beam. The bending moment diminishes as it approaches the supports, and at certain points some of the bars will be available for bending upwards to resist

Fig. 496.

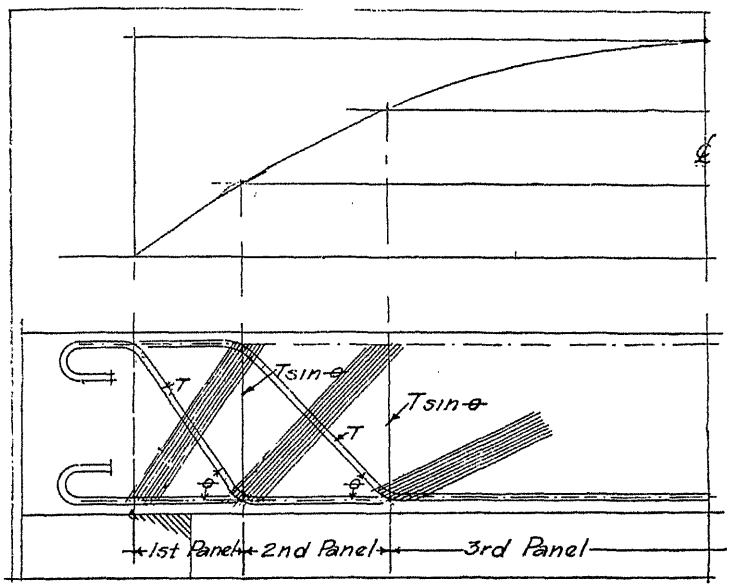


Fig. 497.

Figs. 496—497.

shear. Where this occurs the case becomes similar to a lattice girder, the inclined bars taking the tension stresses, and the compression stresses being taken through the concrete.

In the above diagram, let T be the total tensile force that the bar can take. This can be resolved into horizontal and vertical components. If the angle with the horizontal is θ, the vertical component is T sin θ and this amount is available to resist shear, in combination with the numer-

ically equal vertical component of the diagonal compression band. In a steel lattice girder the bays or panels would be made of a uniform width, and the bars would be increased in sectional area from the centre to the supports. In a ferro beam the inclined bars in each panel are uniform in area, therefore the bays must decrease in width from the centre where the shear is at a minimum to the supports where the stress is at a maximum.

The width of the bays may be determined by setting up the bending stress diagram over the girder. If there are three pairs of bars, as in Fig. 497, divide the altitude into three parts; through these draw horizontals, and at the intersection of these with the graph drop projectors on to the beam. This will give the points where the turn-ups may be made and also the width of the panels.

The width of the panels may be computed in the case of beams under a distributed load by first dividing the bending moment by the number of panels and equating this with the expression for the B at any point.

The points where the bars may be turned up can be determined from the bending moment diagram. In the case of a simply supported beam under a distributed load, the bending moment diagram will be a parabola with an altitude $= \frac{wl^2}{8}$. In the case of a beam with ends fixed, the BM at the centre $= \frac{wl^2}{24}$ and the reverse BM at the ends is $\frac{wl^2}{12}$. These two added $= \frac{wl^2}{8}$. To comply with the Code, the BM is given as $\frac{wl^2}{10}$ both at centre and ends. This is done to cover any possible redistribution of the loads. To set out the diagram (Fig. 498), from any base AB set up a parabola the altitude $= \frac{wl^2}{8}$. Then raise the base till the altitude $= \frac{wl^2}{10}$. From the second base CD project downward from the end C a distance $= \frac{wl^2}{10}$ and draw a third

base EF, on this set up a second parabola altitude $= \frac{wl^2}{8}$. Then any intercept between the base CD and the portions of the curve above and below the base CD will represent the bending moments at every point. (1) When the BM in centre $= \frac{wl^2}{10}$ and (2) when the BM at the ends $= \frac{wl^2}{10}$

Case (3) (p. 756).—This is a continuous T beam with No. 4 bars in tension, only two of which need run right through the bottom of the beam. Two can be stopped and are available for shear resisting purposes (see Fig. 498).

$$S = \frac{W}{2} = \frac{36000}{2} = 18000 \text{ lbs.}$$

To determine where the bars shall be bent up. In the present case only one pair of bars are available. Then divide the altitude of the parabola $\frac{wl^2}{8}$ into two parts. The intersection with the curve gives the point where the bend can be made, this being the point where the M is half its maximum and consequently where one pair of bars can be dispensed with for tensional resistance on the lower side of the beam. This can be calculated as follows: Determine the point where the B is halved

Then $$\frac{wlx - wx^2}{2} = \frac{\frac{wl^2}{8}}{2}$$

cancelling $\frac{w}{2}$ throughout.

$$lx - x^2 = \frac{l^2}{8}$$

$$x^2 - lx + \frac{l^2}{8} = 0.$$

$$\therefore \quad x = \frac{20 \pm \sqrt{400 - \frac{1600}{8}}}{2}$$

$$x = 2{\cdot}9 \text{ or } 17{\cdot}1, \text{ say } 2 \text{ ft. } 10 \text{ ins.}$$

Let one pair of bars be turned up at 2 ft. 10 in. from the support.

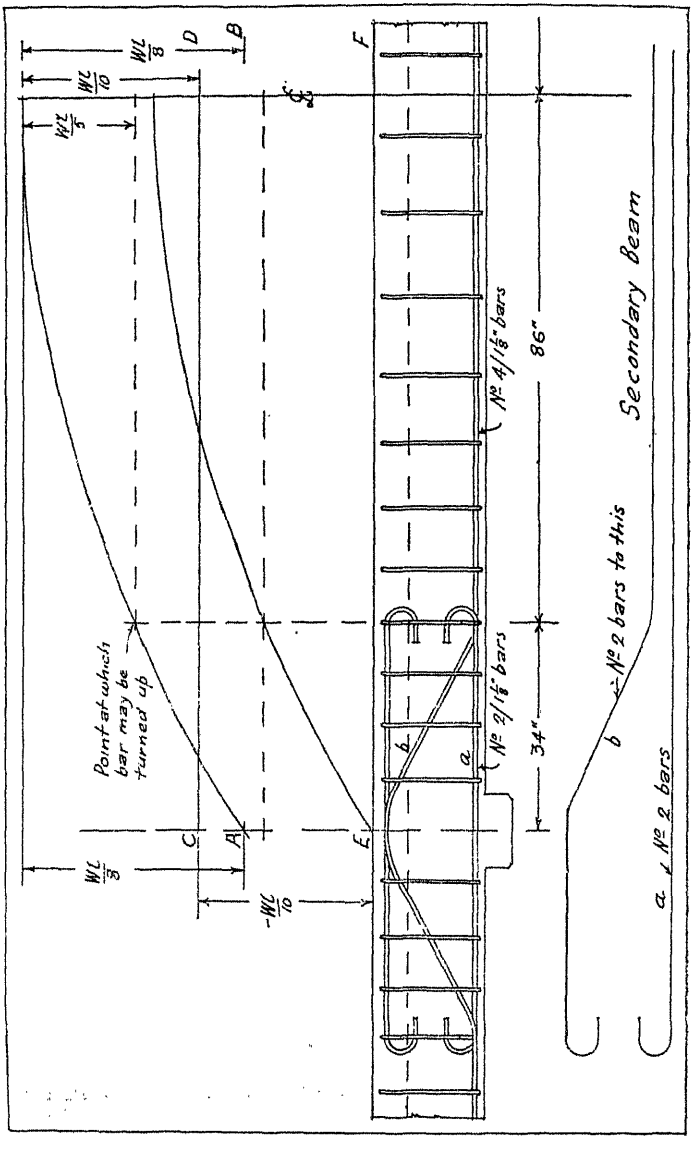

WL/10
WL/8
D
B
WL/5
C
F
Point at which bar may be turned up
WL/8
C
A
E
-WL/10
№ 4 1/8" bars
№ 2 1/8" bars
34"
86"
Secondary Beam
№ 2 bars to this
№ 2 bars
a
b

Fig. 498.

Then the shearing force at the end equals

$$S = \frac{W}{2} = \frac{36000}{2} = 18000 \text{ lbs.}$$

Vertical component of resistance of inclined rods equals T sin θ (see Fig. 496).

$$T \sin \theta = \text{Area of rods} \times t \sin \theta$$
$$= 2 \times 0{\cdot}9940 \times 18000 \times \frac{14}{36} = 13900 \text{ lbs.}$$
$$\text{Resistance of compression band} = 13900 \text{ lbs.}$$
$$27800 \text{ lbs.}$$

There is therefore an excess of resistance for shear. Some stirrups are advisable over the portion between the support and 34 inches. There is still the shear between 34 inches and the centre of the beam to be considered. The S at 34 inches will be

$$= \frac{18000 \times 86}{120} = 12900 \text{ lbs.}$$

Using $\frac{3}{8}$-inch rods with four links

$$p = \frac{A_t s a}{S}$$
$$= \frac{4 \times 0{\cdot}1104 \times 18000 \times 14}{12900}$$
$$= 8{\cdot}6 \text{ inches.}$$

Pillars.—Ferro-concrete pillars may be rectangular, circular, or polygonal in section. The first is most common as it conforms better to architectural and constructional requirements. The formwork required for moulding them is less expensive to prepare. The concrete is reinforced with light steel bars, four or more in number according to the shape of the section, and usually of a combined area of not less than 0·8 nor more than 8·0 per cent. of the gross cross-sectional area of the column. The light steel rods, when subjected to a load, would, if unaided, buckle and burst the protective coating of the concrete. They must be, therefore, securely connected together by binding steel at intervals not greater than 12 inches, and need not be less than 6 inches. This converts each bar into a short column which tends to fail by compression only, and eliminates the tendency to buckle. The volume of trans-

verse reinforcement shall not be less than 0·4 per cent. of the gross volume of the column. The binding also has the effect of confining the enclosed core of concrete and preventing its tendency to crumble or burst under the load. Four rods are usually placed in pillars up to 16 inches in diameter, eight rods in pillars above 16 inches diameter up to 24 inches diameter. The rods are placed as near the outside of the pillar as possible, to obtain the maximum moment of inertia (I).

Cover.—The protection cover is measured from the outside of the steel to the outside of the bar, and must be at least 1 inch or the diameter of the bar.

Column Formulæ.—The accepted theories on pillars give results that are in practice unsatisfactory when applied to ferro-concrete. The conditions for Eulers and the Rankine Gordon formulæ are non-existent, and the findings are too unreliable. Therefore the values given in clause 701 of the Code may be taken to represent general experience based on observation of completed works.

The graph (Fig. 499) shows that in circular and rectangular pillars, the full value of the concrete can be allowed up to a value of 15 for $\frac{l}{d}$ and that the values decrease as a straight line to zero when the $\frac{l}{d}$ is 45.

Where steel and concrete act together in compression, it is necessary, in order that the shortening of the two materials may be equal under load, to limit the stress on the steel to m times the stress in the concrete. In estimating the resistance of the two materials, an equivalent area A_e is computed in terms of the concrete. Thus $A_e = A_c + (m - 1)A_i$ and its resistance $P = c(A_c + (m - 1)A_i)$.

Recent investigations, however, on the behaviour of concrete under stress, have shown the existence of the property of creep. This is a plastic adjustment in the concrete when stressed. This creep in pillars, taken in conjunction with the shrinkage of the concrete, causes the steel to take up more of the stress than it was assumed

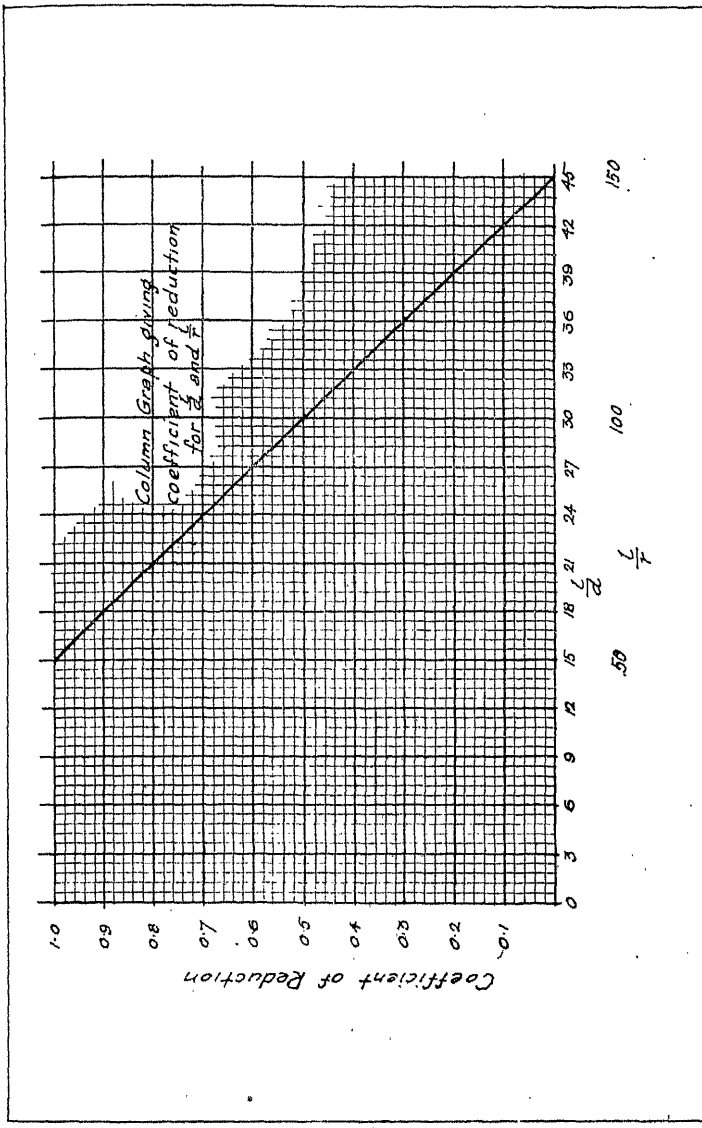

Column Graph giving coefficient of reduction for $\frac{l}{a}$ and $\frac{l}{r}$
Coefficient of Reduction
1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
3
6
9
12
15
18
21
24
27
30
33
36
39
42
45
$\frac{l}{a}$
$\frac{l}{r}$
59
100
150

Fig. 499.

to take, when the stress was determined by the modular ratio. The combined longitudinal contraction is such as to throw a greater stress on the steel. It is within the range of possibility that the steel may become stressed up to the yield point. It cannot, however, go beyond this, as the increased stress would then be taken by the concrete.

The formula employed till recently, in which the resistance of pillars was determined by a steel stress based on the modular ratio, has now been replaced by one in which the strength of the pillars is the sum of the resistances of the steel and the concrete (see Code), *i.e.*, $P = cA_c + tA$.

A_c, the area of the concrete, apparently includes the concrete outside the reinforcement, less area of longitudinal steel.

EXAMPLE.—Taking a pillar 12″ × 12″, 10 feet in height, reinforced with No. 4/⅝-inch-bars. The buckling factor would be $\frac{l}{d} = \frac{120}{12} = 10$. Therefore the coefficient given by the Code would be 1.

Then
$$\begin{aligned} P &= cA_c + tA \\ &= 600 \times 142{\cdot}77 + 13500 \times 4 \times 0{\cdot}3068 \\ &= 85700 + 16600 \\ &= 102300 \text{ lbs.} \end{aligned}$$

Clause 702 of the Code gives a tabulated list of the coefficients that must be used, to determine the reduction in strength when the $\frac{l}{d}$ is any quantity between 15 and 45. It also gives a tabulated statement to arrive at the effective length of the column which varies according to the conditions of restraint at the ends of the columns.

Eccentric Loading.—The determination of the M on a column due to the deflection of the beam attached to it is a very involved question, and owing to the many uncertainties surrounding column calculation, a very exact analysis of the subject is unnecessary. The general principles may be understood from the following considerations. Ferro beams and the columns that support them are monolithic structures, and the joint between them is rigid. Therefore, when the beam is loaded, and deflection ensues, the slope at the end of the beam is imposed on to the end of the column.

In a beam supported only the M at the end is zero, and the slope will be a maximum for the load. In a beam in which the ends are fully restrained the M at the end will reach a maximum and the end slope will be zero. The actual M on the end of a beam will be somewhere between the M for a fully restrained beam and zero when supported only. Where a beam is rigidly fixed to the column as in ferro work, the M will depend on the relative stiffness of the column and beam. The stiffness of any member is the moment of inertia divided by the length of the member. In practice the column is never absolutely rigid, but it always exerts a restraining influence on the beam, causing a reverse M on the latter, and itself becoming subject to the same moment. The resulting slope in the beam and the column will both be equal.

In the case of a beam under a distributed load, fully restrained at the ends, the M would be $\frac{Wl}{12}$. If the column is not sufficiently rigid the M would be less than $\frac{Wl}{12}$, and the work would be divided between the two members. The beam will deflect and develop an end slope which it will impose on the end of the column. The amount of the reverse M taken by the beam and the end of the column, will be in the ratio of the relative stiffness of the two members. The proportion taken at the beam end will be $\frac{S_c}{S_b + S_c}$ and therefore the M at the end of the beam and column will equal

$$M = \frac{Wl}{12} \times \frac{S_c}{S_b + S_c} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (25)$$

The slope at the end of the beam when supported only $= \frac{Wl^2}{24\,EI}$. The slope at the end of the partially restrained beam is

$$\alpha_b = \frac{Wl^2}{24\,EI} \times \frac{S_b}{S_b + S_c}. \quad . \quad . \quad . \quad . \quad . \quad . \quad (26)$$

The slope at the end of the column is

$$\alpha_c = \frac{Wl^2}{2EI} = Wl \times \frac{l_c}{2EI_c} \quad . \quad . \quad . \quad . \quad . \quad . \quad (27)$$

The Wl = the M at the end of the partially restrained beam.

The results of (26) and (27) should be the same, that is, $a_b = a_c$.

The bending moment is divided between the upper and lower columns.

$$\therefore \qquad M = \frac{Wl}{12} \times \frac{S_{c_u}}{S_{c_l} + S_{c_u} + S_b}$$

or using the nomenclature of the Code

$$\underset{\text{at foot of upper col.}}{M} = M_e \frac{K_u}{K_l + K_u + K_b}.$$

EXAMPLE.—A ferro beam 22″ × 10″, 20 feet span, has a load of 70,000 lbs. divided between two point loads of 35,000 lbs. at third points on the beam, $I_b = 19{,}253$ inches.[4] The bending moment M_e at the supports for a single span beam fixed at the ends and loaded as above is $\frac{Wl}{9}$. The beam is joined to pillars 20″ × 20″ fixed at both ends and 12′–0″ long, effective length 9 feet. Upper pillar assumed similar, $I_c = 19{,}320$ inches.

$$\text{Stiffness of col. } \frac{19320}{108} = 179$$

$$\text{,, beam } \frac{19253}{240} = 80$$

$$M = M_e \frac{K_l}{K_u + K_l + K_b} \text{ (see Code)}$$

$$= \frac{Wl}{9} \cdot \frac{179}{179 + 179 + 80}$$

$$= \frac{70000 \times 240 \times 0{\cdot}41}{9}$$

$$= 767000 \text{ lbs. inches.}$$

Then stress due to bending $f = \frac{My}{I}$

$$f = \frac{767000 \times 10}{19320} = 398 \text{ lbs./inch.}^2$$

Axial load, if the column supports two floors, is

$$2 \times 35000 = 70000$$
$$\text{Weight of col. (say)} = \underline{5000}$$
$$75000 \text{ lbs.}$$

$$p_a = \frac{P}{A_e} \text{ where } A_e = A_c + (m - 1)A$$

$$= \frac{75000}{482} = 156 \text{ lbs./inch.}^2$$

Then maximum compressive stress = 398 + 156
= 554 lbs./inch.^{2}

Moment of Inertia.—To determine the moment of inertia of a ferro column by obtaining the equivalent area (Fig. 500). The area of the reinforcement is obtained by using the appropriate modular ratio, *i.e.*, $A_e = A_c +$

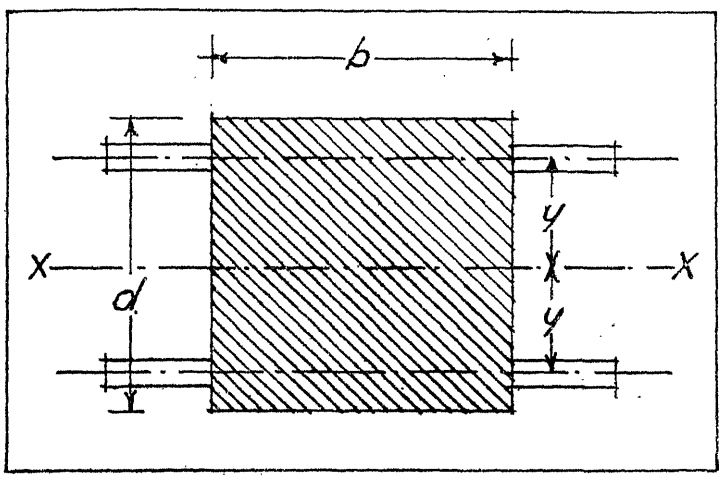


Fig. 500.

$(m - 1)A_t$. From Fig. 500 the hatched portion = $b \times d = A_c$ and $(m - 1)A_t$ = the equivalent area of the steel. The I of the hatched portion about XX is $\frac{bd^3}{12}$. The I of the projected parts = $I_{cg} + (m - 1)A_t \times y^2$. The I_{cg} of bars about their own axis is a very small quantity and may be neglected. Then the I of the section becomes $I = I_c + (m - 1)A_t y^2$.

EXAMPLE.—Take a 12″ × 12″ column, with No.

$4/\frac{3}{4}$ inch diameter bars. The core to the outside of the bars is 10″ × 10″, $m = 18$.

$$I = \frac{12 \times 12^3}{12} + 17 \times 4 \times 0{\cdot}4418 \times (4\tfrac{5}{8})^2$$
$$= 1728 + 640 = 2368 \text{ inches}^4.$$

To determine the Moment of Inertia for a T beam, it is first necessary to find the position of the NA (see example of T beam, p. 745). Referring to Fig. 489,

$$I = \left(\frac{bd_s^3}{12} + bd_s y_1^2\right) + mA_t y_2^2.$$

where $y_1 = n, y_2 = d - n$

The I of the bars about their own axis is a negligible quantity.

$$\therefore \quad I = \left(\frac{39 \times 4{\cdot}25^3}{12} + 39 \times 4{\cdot}25 \times 6{\cdot}32^2\right) + (18 \times 3{\cdot}98 \times 9{\cdot}68^2)$$
$$= 13569 \text{ inches.}^4$$

EXAMPLE.—The following is an example of the calculations required for a building of the office type, of the dimensions and construction shown on Figs. 501 and 502. There are to be four storeys, each 12 feet in height. The super load to be 100 lbs. per square foot. The span of secondary beams 20 feet; the span of main beams 18 feet. Weight of concrete to be 144 lbs. per cubic foot.

The slabs and beams in ferro work are of the continuous type. The moments on them and their reactions should be determined from the Theorem of Three Moments explained in the chapter on Girders, which should be carefully studied for application in particular cases. For general work an exact analysis is superfluous, as any rearrangement of the loads would alter the values of the stresses on the various members. Therefore the values given in clause 602b of the Code would cover most cases and can be adopted. It should be noted that maximum moments in beams usually occur when alternate bays of a structure only are loaded; also that the beams on the unloaded bays may develop tension on the top side, and therefore sufficient steel should be placed there to provide for that contingency.

Fig. 501.

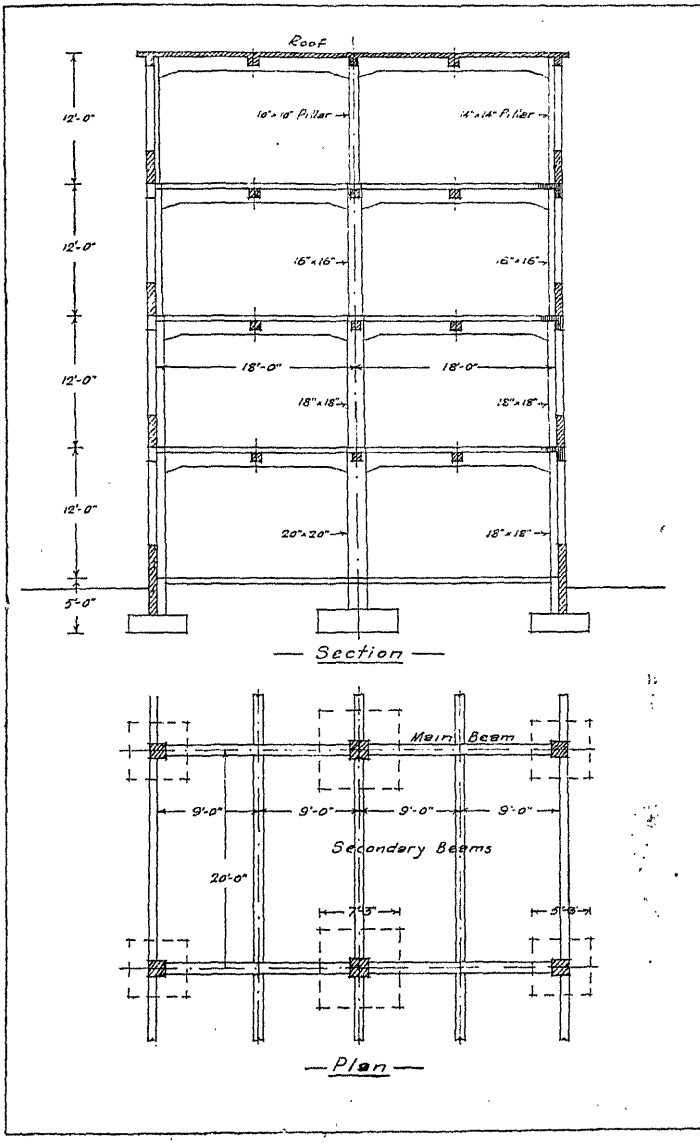


Fig. 502.

Figs. 501—502.

Floor Slab.—The slab is 20′ × 9′.

$$\text{Ratio } \frac{l_y}{l_x} = \frac{20}{9} = 2{\cdot}22$$

Therefore factor for reduction of bending moment (Code para. 602, Table 5, Case (*a*)).

$$z_x = 0{\cdot}757 + \tfrac{2}{5}(0{\cdot}869 - 0{\cdot}757)$$
$$= 0{\cdot}802$$

The reinforcement will be across in the direction z_x. Then the load per super foot will equal

Superload	...	...	...	100 lbs.
Dead load, say $\frac{5}{12} \times 144$			...	60 ,,
Finishings	...	...	...	20 ,,
				180 lbs. per square foot.

Taking a strip 1 foot wide W = 9 × 180 × 1,620 lbs.

$$\text{From Code, } M_x = z_x \cdot \frac{Wl}{8} = \frac{0{\cdot}802 \times 1620 \times 108}{8}$$
$$= 17500 \text{ lbs. inches.}$$

$$\text{Thickness of slab } d = \sqrt{\frac{M}{Qb}} = \sqrt{\frac{17500}{137{\cdot}6 \times 12}}$$
$$= 3{\cdot}26, \text{ say } 3{\cdot}5 \text{ inches.}$$

Use ½-inch dia. rods, A = 0·1963 square inches.

$$\text{Pitch of rods, } b = \frac{A_t}{rd} = \frac{0{\cdot}1963}{0{\cdot}00893 \times 3{\cdot}5}$$
$$= 6{\cdot}28, \text{ say } 6 \text{ inches.}$$

Add ½ inch for cover.

$$\text{Total thickness} = 3{\cdot}5 + 0{\cdot}25 + 0{\cdot}5$$
$$= 4{\cdot}25 \text{ inches}$$

Use ⅜ inch distributing bars

pitch = 4 × effective depth of slab.

$$= 4 \times 3{\cdot}5 = 14 \text{ inches.}$$

Each bar should be cranked upward at one end at a point approximately $\frac{l}{5}$ from the support. Both ends should be continued beyond the centre of the supports a distance equal to 30 *d* and hooked to provide the necessary bond (see Figs. 503 and 504).

Secondary Beams.—The secondary beams support a portion of the slab 9′ × 20′. Let the effective depth be

taken as 15 inches and the breadth as 10 inches. The lever arm $a = 0{\cdot}857 \times 15'' = 12{\cdot}9$ inches. The loads will then be as under :

Slab $= 9 \times 20 \times 180 = 32440$
Rib say $= 20 \times 1 \times 1 \times 144 = 2880$

35320 lbs. $= 15{\cdot}76$ tons

As the beams will be continuous, take the moments as $\frac{Wl}{10}$ in centre of the span and $-\frac{Wl}{10}$ at the supports (see Fig. 506).

Then $$M = \frac{Wl}{10} = \frac{35320 \times 240}{10}$$
$$= 847680 \text{ lbs. inches.}$$
$$A_t = \frac{M}{fa} = \frac{850000}{18000 \times 12{\cdot}9} = 3{\cdot}6 \text{ square inches}$$

Use No. 6/$\frac{7}{8}$-inch rods.

Compression Flange

$$A_c = \frac{M}{fa}. \text{ Assume } f = \frac{750}{2} = 375 \text{ lbs./inches.}^2$$
$$A_c = \frac{847680}{350 \times 12{\cdot}9} = 175 \text{ square inches.}$$
$$\text{Width of slab} = \frac{175}{4{\cdot}25} = 41 \text{ inches}$$

The maximum width of slab permissible for this type of beam (see Code, section 6*g*) equals the breadth of the rib plus four times the thickness of the slab. Thus :

$$10'' + (12 \times 4{\cdot}25'') = 61 \text{ inches}$$

Then to determine the position of the " Neutral Axis " take moments about the top of the slab.

The equivalent area of the steel $= A_t \times m$
$= 3{\cdot}6 \times 18 = 65$ square inches.

Then $$bd_s\frac{d_s}{2} + mA_td = An$$
$$41 \times 4{\cdot}25 \times 2{\cdot}125 + 18 \times 3{\cdot}6 \times 15 = 240n$$
$$n = 5{\cdot}6 \text{ inches}$$

Distance from neutral axis to centre of slab $= 3{\cdot}48$ inches

Then the average resistance in the compressional flange $= \frac{750 \times 3{\cdot}48 \text{ in.}}{5{\cdot}6}$
$= 467$ lbs. per square inch.

The assumed resistance was 375 lbs./square inch. If the permissible width of flange of 61 inches were taken, the stress would of course be much less.

In the centre of the beam the slab was made use of to form the top flange of the beam and take the compression stresses. At the points of support the stresses are reversed and the flange is no longer available for this purpose and the rib is altogether inadequate. There are two methods of providing the required compressional resistance; first, by forming a haunch, thus deepening the beam at this point; or secondly by providing sufficient reinforcement, making a doubly reinforced beam at this point. Thus, in this case, one pair of rods is run through in each beam. The distance can be determined by the stress diagram. This makes four rods and the addition of two more will provide the necessary resistance. There is rarely sufficient depth in the main beams to allow of the formation of a haunch in the secondary beams.

Shearing.—At the supports $S = \frac{W}{2} = \frac{35320}{2}$

$= 17{,}660$ lbs.

Two pairs of bars are available for bending up. Add one pair at the support. This divides the half length into four panels. Set out a diagram of the bending moments as explained in the example on p. 769. In this case the moments are taken as $\frac{Wl}{10}$ at the centre of the beam and at the supports. Divide the altitude of the upper parabola $\frac{Wl}{8}$ into four parts and draw lines parallel to the base to intersect the curve; from these points drop projectors on to the base line. These will be the points where the bars may be bent upwards (see Fig. 506).

Resistance to shear of one pair $\frac{7}{8}$ inch dia. bent up bars is T sin θ (see Fig. 496).

1st panel. $T \sin \theta = 2 \times 0{\cdot}6013 \times 18000 \times \frac{12{\cdot}9}{20{\cdot}6} = 13500$

Resistance of compression band $= 13500$

Total 27000 lbs.

This is in excess of requirements.

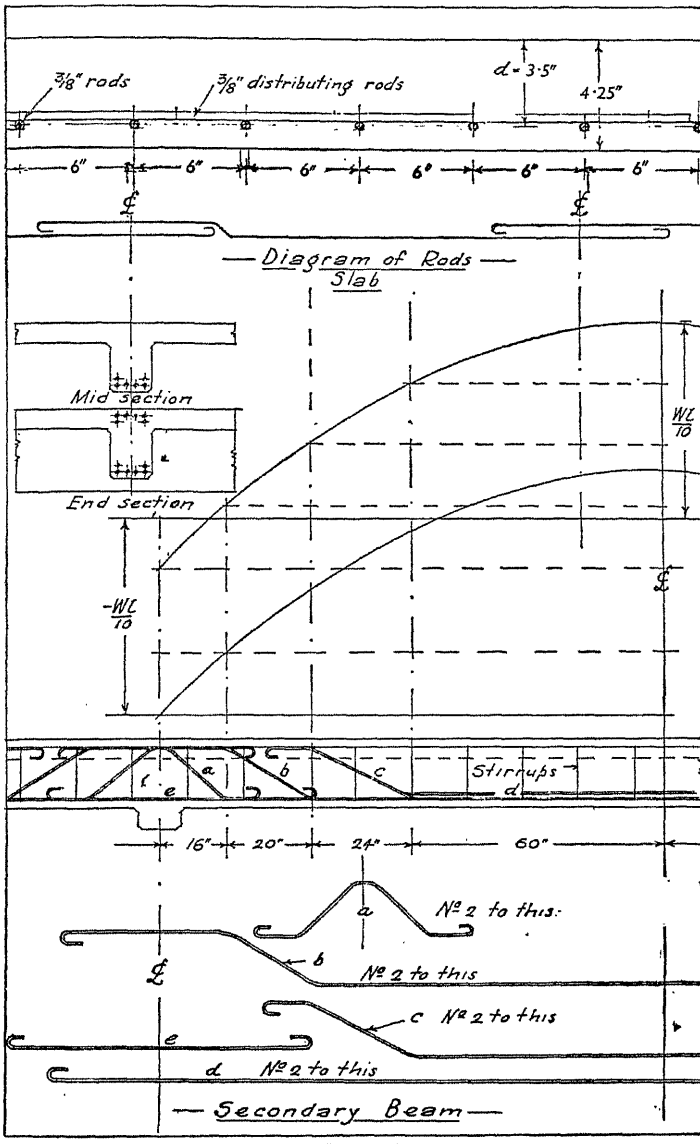


Fig. 503.

Fig. 504.

Fig. 507.

Fig. 506.

Fig. 505.

Fig. 508.

Figs. 503—508.

Shear at 104 inches from centre $= \frac{17660 \times 104}{120}$ = 15300 lbs.

2nd panel. $T \sin \theta = 2 \times 0{\cdot}6013 \times 18000 \times \frac{12{\cdot}9}{23{\cdot}8}$ = 11700

Resistance of compression band = 11700

Total 23400 lbs.

This is in excess of requirements.

Shear at 84 inches from centre $= \frac{17660 \times 84}{120}$ = 12360 lbs.

3rd panel. $T \sin \theta = 2 \times 0{\cdot}6013 \times 18000 \times \frac{12{\cdot}9}{27{\cdot}2}$ = 10300

Resistance of compression band = 10300

Total 20600 lbs.

This is in excess of requirements.

Shear at 60 inches from centre $\frac{17660 \times 60}{120}$ = 8830 lbs.

$$S = \frac{S_v}{ab} = \frac{8830}{12{\cdot}9 \times 10} = 68{\cdot}5 \text{ lbs./inch.}^2$$

Permissible shear stress in concrete is 75 lbs./inches ^{2}, so that no stirrups would be required. It would, however, be necessary to place stirrups at a pitch not exceeding a, the arm of the resistance moment (see Figs. 505 and 508).

Main Beams.—These carry concentrated loads at a central point made up of the superimposed load, consisting of the reactions from the secondary beams, and, in addition, a distributed load, being the weight of the beam. Select a beam 20 inches effective depth and breadth 10 inches.

Central point load = 15·76 tons = 35320 lbs.
Weight of rib, say = 2 tons = 4480 lbs.

39800 lbs.

The beam being continuous over two spans there will be a reverse moment over the central support which will be a maximum when both bays are loaded. The greatest bending moment near the middle of the span will occur when one bay only is loaded, in which case there will be a negative moment in the unloaded span, resulting in a tensional stress in the top of the beam, for which reinforcement must be provided.

The expression for the bending moment over the central support is

$$M_2 = -\frac{3W_1 l}{32} \text{ if one bay is loaded}$$

and $$M_2 = -\frac{3l}{32}(W_1 + W_2) \text{ if both bays are loaded.}$$

where W_1 and W_2 are the concentrated loads. In this case $W_1 = W_2$.

The calculation of M_2 for the first case is given below (see p. 613, similar examples in the Chapter on Girders).

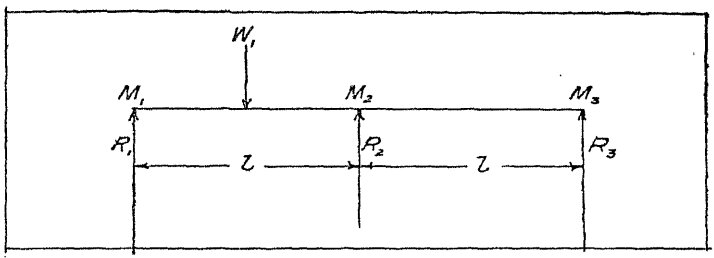


Fig. 508*a*.

Refer to Fig. 508*a*. Write down general expressions for the bending moment in the three sections of the beam between M_1 and W_1, W_1 and M_2, M_2 and M_3.

$$EI\frac{d^2y}{dx^2} = R_1 x \quad \left| \begin{array}{l} -W_1\left(x - \frac{l}{2}\right) \\ \end{array} \right| + R_2(x - l) \quad \left| \begin{array}{l} \\ \end{array} \right| \quad \ldots \quad (1)$$

$$EI\frac{dy}{dx} = R_1\frac{x^2}{2} + C \quad \left| \begin{array}{l} -W_1\frac{\left(x - \frac{l}{2}\right)^2}{2} \\ \end{array} \right| + \frac{R_2(x-l)^2}{2} \quad \left| \begin{array}{l} \\ \end{array} \right| \quad \ldots \quad (2)$$

$$EIy = R_1\frac{x^3}{6} + Cx + D \quad \left| \begin{array}{l} -W_1\frac{\left(x - \frac{l}{2}\right)^3}{6} \\ \end{array} \right| + \frac{R_2(x-l)^3}{6} \quad \left| \begin{array}{l} \\ \end{array} \right| \quad \ldots \quad (3)$$

When $x = 0$, $y = 0$ $\quad \therefore D = 0$

,, $x = l$, $y = 0$

Substitute $x = l$ in (3).

$$0 = R_1\frac{l^3}{6} + Cl - \frac{W_1 l^3}{48} \quad \ldots \quad (4)$$

Also when $x = 2l$, $y = 0$.

Substitute $x = 2l$ in (3).

$$0 = \frac{8R_1 l^3}{6} + 2Cl - \frac{27\,W_1 l^3}{48} + \frac{R_2 l^3}{6} \quad \ldots \quad (5)$$

Fig. 509.

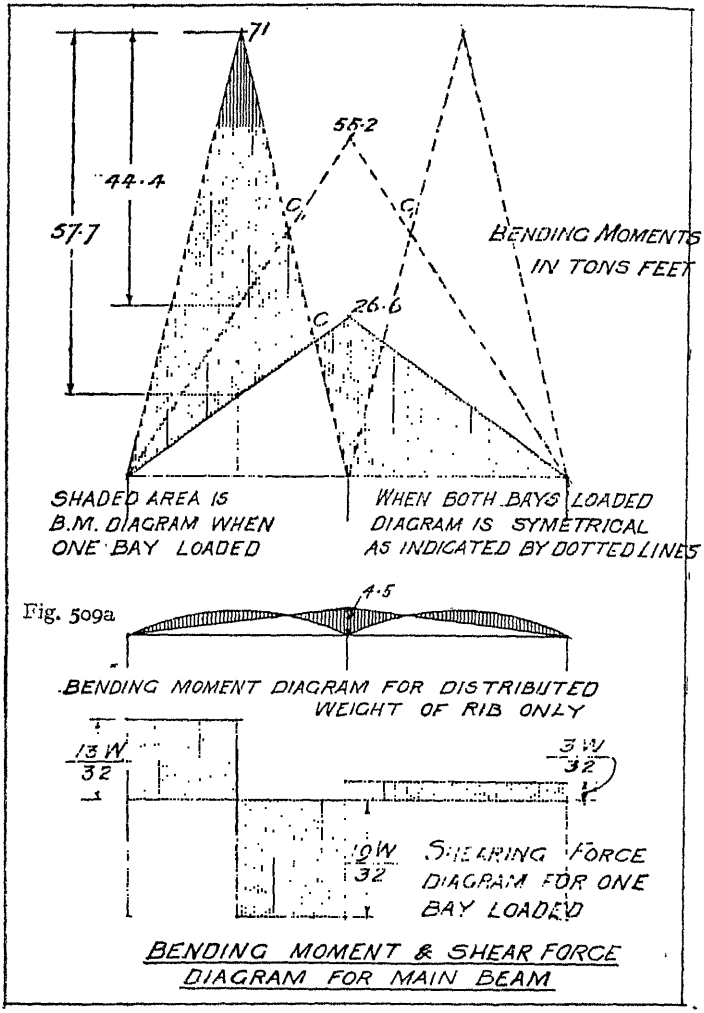


Fig. 509a

Fig. 510.

Figs. 509—510.

Multiply equation (4) by 2, and (5) by unity and eliminate C by subtraction.

(5) × 1 $$0 = \frac{8R_1l^3}{6} + 2Cl - \frac{27W_1l^3}{48} + \frac{R_2l^3}{6}$$

(4) × 2 $$0 = \frac{2R_1l^3}{6} + 2Cl - \frac{2W_1l^3}{48}$$

Subtracting

$$0 = R_1l^3 - \frac{25W_1l^3}{48} + \frac{R_2l^3}{6}$$

or $$0 = R_1 - \frac{25W_1}{48} + \frac{R_2}{6} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (6)$$

Bending moment = 0 when $x = 2l$.

Substitute in (1)

$$0 = 2R_1l - \frac{3W_1l}{2} + R_2l$$

or $$0 = 2R_1 - \frac{3W_1}{2} + R_2 \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (7)$$

Multiply equation (6) by 2 and (7) by unity and eliminate R_1 by subtraction.

(7) × 1 $$0 = 2R_1 - \frac{3W_1}{2} + R_2$$

(6) × 2 $$0 = 2R_1 - \frac{25W_1}{24} + \frac{R_2}{3}$$

Subtracting

$$0 = -\frac{11W_1}{24} + \frac{2R_2}{3}$$

$$\therefore \quad R_2 = \frac{11W_1}{16}$$

Substitute for R_2 in (6)

$$0 = R_1 - \frac{25W_1}{48} + \frac{11W_1}{96}$$

$$\therefore \quad R_1 = \frac{13W_1}{32}$$

When $x = l$, $EI\frac{d^2y}{dx^2} = M_2$

Substitute in (1)

$$\therefore \quad M_2 = R_1l - W_1\left(l - \frac{l}{2}\right)$$

$$= \frac{13W_1l}{32} - \frac{W_1l}{2}$$

$$M_2 = -\frac{3W_1l}{32}$$

And $$R_3 = W_1 - R_1 - R_2$$

$$R_3 = -\frac{3W_1}{32}$$

The value of M_2 when both bays are loaded can be calculated in a similar manner.

Value of M_2 for one bay loaded

$$M_2 = -\frac{3 \times 15{\cdot}76 \times 18}{32} = -26{\cdot}6 \text{ tons feet.}$$
$$= -715{,}000 \text{ lbs. inches.}$$

Value of M_2 for both bays loaded.

$$M_2 = -\frac{3 \times 18 \times 31{\cdot}52}{32} = -53{\cdot}2 \text{ tons feet.}$$
$$= -1{,}430{,}000 \text{ lbs. inches.}$$

The maximum bending moment in the centre for beams simply supported at ends and with a concentrated load at the centre is $\frac{Wl}{4}$. Assume that each bay is simply supported at ends. Then bending moment M at mid span is

$$M = \frac{Wl}{4} = \frac{15{\cdot}76 \times 18}{4} = 71 \text{ tons feet.}$$

Draw the bending moment diagram (Fig. 509) in a manner similar to comparable examples in the Chapter on Girders (see Fig. 444). Taking the case of one bay loaded, set out the negative bending moment 26·6 tons ft. obtained above and join this point to the free ends of beam where moment is zero. Then set out the bending moment diagram for the loading as if each individual bay were simply supported at the ends: set up 71 tons ft. at centre of loaded span and join to zero at ends of span. The final diagram for the loading is that part shown hatched in the diagram, positive to one side of the point of contraflexure C, negative on the other.

The diagram shown by the dotted lines is that obtained when both bays are loaded. Notice that there are two points of contraflexure (C_1).

Draw a separate bending moment diagram (Fig. 509*a*) for the distributed load due to the weight of rib. See the

Theorem of Three Moments (p. 600, Fig. 438) for the value of M_2.

$$M_2 = -\frac{wl^2}{8} = -\frac{Wl}{8} = \frac{2 \times 18}{8}$$

$$= -4{\cdot}5 \text{ tons ft.}$$

$$= 120960 \text{ tons inches.}$$

Set up parabolæ over each individual span representing the bending moment diagrams for distributed loads, where $\frac{Wl}{8}$ is the maximum bending moment at mid span for beams simply supported at ends. The finished diagram showing positive and negative moments is shown hatched.

The final values of bending moment for computation are obtained by adding together algebraically the corresponding values obtained from Figs. 509 and 509*a*.

BM at centre of beam $= 57{\cdot}7 + 2{\cdot}25 = 59{\cdot}95$ tons ft.
$= 1{,}610{,}000$ lbs. inches.

BM over central support $= 53{\cdot}2 + 4{\cdot}5 = 57{\cdot}7$ tons ft.
$= 1{,}550{,}000$ lbs. inches.

Negative moment at centre of the unloaded bay $= 13{\cdot}3 - 2{\cdot}25 = 11{\cdot}05$ tons ft.
$= 297{,}000$ lbs. inches.

Tensional Reinforcement.

Assume $a = 0{\cdot}857d \qquad \therefore\ d = 17{\cdot}1$

$$A_t = \frac{M}{ta} = \frac{1610000}{18000 \times 17{\cdot}1} = 5{\cdot}2 \text{ square inches.}$$

Use No. 8/$1\frac{1}{16}$-inch dia. rods.

Compressional flange. Assume $c = \frac{750}{2} = 375$ lbs./square inch.

$$A_c = \frac{M}{ca} = \frac{1610000}{375 \times 17{\cdot}1} = 250 \text{ square inches.}$$

Width of flange $= \frac{A_c}{ds} = \frac{250}{4{\cdot}25} = 58{\cdot}8$ inches.

It would be advisable to place an extra rod in the slab between each of the distributing bars for the width of the flange.

Ratio of Reinforcement

$$r = \frac{A_t}{bd} = \frac{5{\cdot}2}{58{\cdot}8 \times 20} = 0{\cdot}0044$$

Depth of "n" below Top of Beam.—Take moments about the top edge of the slab.

$$An = bd_s\frac{d_s}{2} + mA_td$$

$$344n = (58{\cdot}8 \times 4{\cdot}25 \times 2{\cdot}13) + (18 \times 5{\cdot}2 \times 20)$$

$$n = \frac{2405}{344} = 7{\cdot}0 \text{ inches.}$$

Moment of Inertia of Beam

$$I = \left(\frac{bd^3}{12} + A_cy^2\right) + \left(A_ty^2m\right) \text{ (see Fig. 365)}$$

$$= \left(\frac{58{\cdot}8 \times 4{\cdot}25^3}{12} + 250 \times 4{\cdot}88^2\right) + \left(5{\cdot}2 \times 13^2 \times 18\right)$$

$$\left\{y = (7 - 4{\cdot}25) + \frac{4{\cdot}25}{2} = 4{\cdot}88\right\}$$

$$I = 22180 \text{ inches.}^4$$

Haunch. As there will be no compressional flange at the end of the beam to resist the compression, increase the depth at that part by forming a haunch.

Let the haunch be 30 inches deep with an effective depth of 28·5 inches (Fig. 513). There will be six $\frac{5}{16}$-inch rods run through in the top or tensional side of the beam. The reverse moment at the outside pillar is unknown, until the relative stiffnesses are taken into account, but at this point it may be taken as being equal to the moment on the inside pillar; which in this case equals 1,550,000 lbs. inches.

Area of six $\frac{15}{16}$-inch diameter rods = 4·1418 square inches.

$$M_s = A_tta = 4{\cdot}1418 \times 18000 \times 24$$
$$= 1790000 \text{ lbs. inches.}$$

To determine if the concrete area is sufficient it is necessary to find the value of "r" and "n."

$$r = \frac{A_t}{bd} = \frac{4{\cdot}1418}{10 \times 28{\cdot}5} = 0{\cdot}0145$$

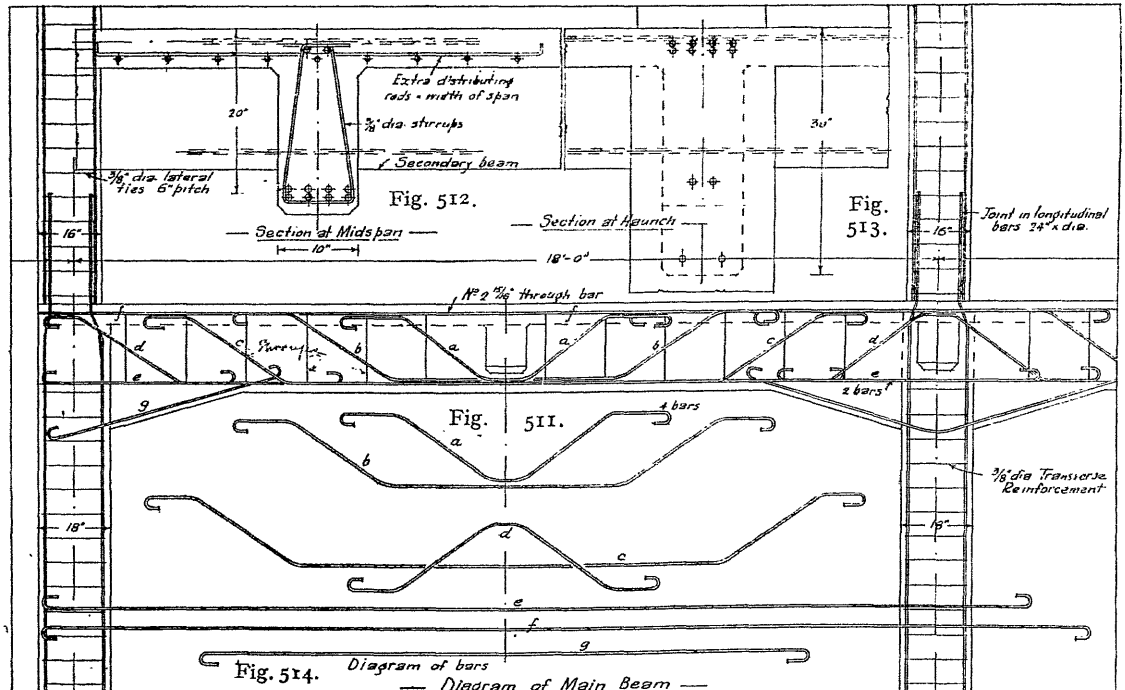
$$n_1 = \sqrt{r^2m^2 + 2rm} - rm$$

$$= \sqrt{0{\cdot}0145^2 \times 18^2 + 2 \times 0{\cdot}0145 \times 18} - 0{\cdot}0145 \times 18$$

$$= 0{\cdot}5$$

$$= n_1d = 0{\cdot}5 \times 28{\cdot}5$$

$$n = 14{\cdot}25 \text{ inches.}$$


Extra distributing rods = width of span
20"
⅜" dia. stirrups
Secondary beam
⅜" dia. lateral ties 6" pitch
Fig. 512.
Section at Midspan
10"
Section at Haunch
18'-0"
30"
Fig. 513.
Joint in longitudinal bars 24" x dia.
16"
No 2 1⅛" through bar
Stirrups
a
b
c
d
e
f
g
2 bars
4 bars
Fig. 511.
⅜" dia Transverse Reinforcement
18"
Fig. 514.
Diagram of bars
Diagram of Main Beam

Figs. 511—514.

Then the concrete area $= 14{\cdot}25 \times 10 = 142{\cdot}5$ square inches

and $$a = 28{\cdot}5 - \frac{14{\cdot}25}{3} = 23{\cdot}75 \text{ inches}$$

$$M_c = A_c c a. \qquad \text{Let } c = 400 \text{ lbs.}$$
$$= 14{\cdot}25 \times 10 \times 400 \times 23{\cdot}75$$
$$= 1350000 \text{ lbs. inches.}$$

This is 200,000 lbs. inches under the requirements and some steel would be required. If No. 2/$\frac{15}{16}$-inch bars are placed a distance "a" from the centre of tension there would be ample resistance in compression.

Tension in Top of Beam.—When one bay only is loaded there will be a negative stress in the top of the adjoining bay, which will reach a maximum near the centre of the beam (see diagram 509). In this case it is 297,000 lbs. inches.

$$T = \frac{M}{a} = \frac{297000}{17{\cdot}1}$$
$$= 17800 \text{ lbs.}$$

Then $$A_t = \frac{W}{t} = \frac{17800}{18000} = 0{\cdot}99 \text{ square inches.}$$

Use No. 2/$\frac{15}{16}$-inch diameter rods to run through the full length of the beam near the top.

Shear.—The maximum shear for the central load with one bay loaded will be at the internal pillar and equals (see Fig. 510).

$$S = \frac{19}{32}W = \frac{19 \times 15{\cdot}76}{32} = 9{\cdot}4 \text{ tons.}$$
$$= 21100 \text{ lbs.}$$

Shear for distributed load

$$S = \frac{10W}{8} = \frac{10 \times 2}{8} = 2{\cdot}5 \text{ tons.}$$
$$= 5600 \text{ lbs.}$$

Total $S = 21100 + 5600 = 26700$ lbs.

See Fig. 510. The shear at the outside pillars will not be so much but for the sake of simplicity arrange the bars symmetrically. Divide each half beam into four panels.

The shear in the four panels will be the same and will equal 26,000 lbs. The resistance to shear of each pair of

bent up bars ($2/\frac{15}{16}$ inches, area 0·6903 inch^2 each) is T sin θ (see Fig. 496).

$$T \sin \theta = 2 \times 0{\cdot}6903 \times 18000 \times \frac{17{\cdot}1}{32} = 13300$$

Resistance of compression band $= 13300$

Total $= 26600$ lbs.

This is satisfactory but it will be advisable to place stirrups at distances not exceeding "*a*."

Pillars.—The general procedure in the computation of pillars has been explained. For continuous pillars running through several stories, it will be necessary first to determine the loads at the different levels. Line diagrams should be prepared to show these. They consist of the reactions of all beams attached to the pillars plus the weight of the pillar. Any extra stress due to unbalanced loads must be determined and added to the total of the axial loads. This total must be within the allowable stresses for the mix of concrete employed.

In selecting the sections to be employed attention should be given to the cost of the formwork. If it is possible to employ the same external dimensions through more than one storey, a considerable saving can be effected, as the same moulds can be used without any alterations.

Whereas in the present case there is a symmetrical arrangement of the beams on the internal pillars, the loads can be taken as axial; any extra stress due to the loading of alternate bays will be slight and can in most cases be neglected. In estimating the loads on the columns let the roof be computed to take the same loads as the floors. Tabulate the loading conditions and the properties of the pillars to facilitate checking.

In the case of the external pillars anything from about one-quarter to three-quarters the stress will be due to bending. For this the M on the beam, the I, and the stiffness of the beam must be computed. The moment at the end of the beam, calculated as for fixed ends, is required. This for a central load is (p. 570) :—

$$\frac{Wl}{8} = \frac{39800 \times 216}{8}$$

$$= 1075000 \text{ lbs. inches.}$$

$$\text{The I} \quad = 22180 \text{ inches}^4$$

$$\text{,, } K_b = \frac{22180}{216} = 102.$$

The equivalent area of the column

$$= A_e = A_c + (m - 1) A_t$$
$$= A_c + (18 - 1)A_t = A_c + 17A_t$$

The columns cannot be computed direct but must be determined by trial. To facilitate this process for axial loading a table of standard columns is given with their resistance and moments of inertia.

STANDARD FERRO PILLARS.

Dimension.	Vertical rods.	R. of concrete.	R. of steel.	Total.	Moment of inertia.
inches.	inches.	lbs.	lbs.	lbs.	inches.^{4}
6 × 6	4 $\frac{1}{2}$″	21130	10000	31730	149
8 × 8	4 $\frac{1}{2}$″	37930	10600	48530	442
10 × 10	4 $\frac{5}{8}$″	59200	16570	75830	1118
12 × 12	4 $\frac{3}{4}$″	85340	23860	109200	2368
14 × 14	4 $\frac{7}{8}$″	116160	32470	148630	4464
16 × 16	4 1″	151710	42410	194120	7710
18 × 18	8 $\frac{3}{4}$″	192280	47710	239990	12240
20 × 20	8 $\frac{7}{8}$″	237110	64940	302050	19320
22 × 22	8 1″	286630	84820	371450	29130
24 × 24	8 1″	341830	84820	426650	39420

The dimensions include the cover.

The cover in each case = 1 inch.

The resistances are the permissibles in the concrete $A_c c$ and the steel $A_t t$ ($t = 13500$).

The moment of inertia $= I = I_c + (m - 1)A_t y^2$.

External Pillars.—The calculations for the external pillars are shown in the table (p. 782). The loading on the pillars is made up as shown below. The external secondary

beams are assumed to carry a 9-inch brick parapet 3 feet high at the roof level and at the lower floor levels the same weight is taken as an allowance for the window back and window.

Roof level :—

Weight of parapet	$= 3' \times 9'' \times 112$ lbs.	$= 5040$
Weight of floor slab	$= \frac{1}{2} \times 32440$ (p. 767)	$= 16220$
Weight of girder (say)		$= 2240$
		23500
	Reaction	$= 11750$ lbs.

Reaction from main beam (p. 770) $= \frac{1}{2}(35320 + 4480)$ $= 19900$ lbs.

Third floor level :—

Axial load from above $= (2 \times 11750) + 19900$	$= 43400$
Reaction from secondary and main beams	$= 43400$
Weight of column $= \frac{14'' \times 14''}{144} \times 12' \times 144$ lbs.	$= 2350$
Total	$= 89150$ lbs.

and so on as in table for lowest levels.

Where possible it is well to avoid too many changes of section or variation in diameter of rod. This will effect economy in both labour and the material used as shuttering, and economy and simplicity in the reinforcement.

Internal Pillars.—The calculations for the internal pillars are shown in the table on p. 783. Two points which should be kept in mind are : (1) that the percentage of reinforcement falls within the limits set in the Code, para. 701 (*a*), and (2) that the ratio of length to least dimension be calculated, and considered in the light of the requirements set out in the table in the Code, para. 702 (*a*), (ii).

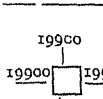
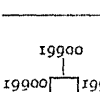
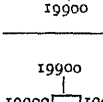
Foundations.—The computations based on a careful analysis of the distribution of the stresses induced by placing a pillar in the centre of a square slab are too involved and cumbersome for general use. The proportions of the slab based on the following assumptions have proved satisfactory in ordinary practice. (1) The lateral dimensions of the slab are determined from the formula

$$A = \frac{P}{p}.$$

EXTERNAL PILLARS

Storey.	Loading. lbs.	Pillar $A_c = A_c + (m-1)A_t$	No. of Rods. Area.	Effective Length.	Moment of Inertia ins.⁴	$K_l = \frac{Ic}{lc}$	$K_b = \frac{Ib}{lb}$	Bending Stress $M = M_c \frac{K_l}{K_l + K_u + K_b}$	Axial Stress $p = \frac{P}{A_c}$	Total Stress.
Roof	11750 / 19900 / 11750 = 43400	14″ × 14″ = 226 sq. ins.	4/¾″ rods, A 1·76 sq. ins.	l = 144″	4148	4148 / 144 = 29	102	1075000 × 29/(29 + 102) = 238000 lbs. ins. = My/I = 238000 × 7/4148 = 402 lbs./in.²	43400/226 = 192 lbs./in.²	402 + 192 = 594 lbs./in.²
3rd F.										
3rd F.	11750 / 19900 / 11750 = 43400 + 43400 + 2350 = 89150	18″ × 18″ = 384 sq. ins.	8/¾″ rods, A 3·53 sq. ins.	0·75l = 108″	12240	12240 / 108 = 113	102	1075000 × 113/(113 + 29 + 102) = 498000 lbs. ins. = My/I = 498000 × 9/12240 = 367 lbs./in.²	89150/384 = 232 lbs./in.²	367 + 232 = 599 lbs./in.²
2nd F.										
2nd F.	11750 / 19900 / 11750 = 89150 + 43400 + 3890 = 136440	18″ × 18″ = 431 sq. ins.	8/1″ rods, A 6·28 sq. ins.	0·75l = 108″	14756	14756 / 108 = 137	102	1075000 × 137/(137 + 113 + 102) = 418000 lbs. ins. = My/I = 418000 × 9/14756 = 255 lbs./in.²	136440/431 = 317 lbs./in.²	255 + 317 = 572 lbs./in.²
1st F.										
1st F.	11750 / 19900 / 11750 = 136440 + 43400 + 3890 = 183730	18″ × 18″ = 431 sq. ins.	8/1″ rods, A 6·28 sq. ins.	204″	14756	14756 / 204 = 72	102	1075000 × 72/(72 + 137 + 102) = 249000 lbs. ins. $f = \frac{My}{I}$ = 249000 × 9/14756 = 152 lbs./in.	183730/431 = 427 lbs./in.²	152 + 427 = 579 lbs./in.²
Found										
Founds	183730 + 5520 = 189250									

INTERNAL PILLARS

Storey.	Loading. lbs.	Pillar.	No. of Rods. Area.	Effective Length.	R. of Concrete. lbs.	R. of Steel. lbs.	Total resistance. lbs.
Roof	 79600	10″ × 10″	4/⅜″ rods A 1·77 sq. ins.	144″	58940	23860	82800
3rd F.							
3rd F.	 79600 79600 1200 160400	16″ × 16″	4/1″ rods A 3·14 sq. ins.	0·75l 103″	151710	42410	194120
2nd F.							
2nd F.	 100400 79600 3100 243100	18″ × 18″	8/⅜″ rods A 3·53 sq. ins.	0·75l 108″	192280	47710	239990
1st F.							
1st F.	 243100 79600 3900 326600	20″ × 20″	8/1″ rods A 6·28 sq. ins.	l 204″	236230	84820	321050
Found							
Founds	326600 6800 333400						

(2) That the depth or thickness of the slab is determined (*a*) by shearing through two vertical planes parallel to the sides of the pillar, (*b*) the depth may be determined by considerations of bending, and (*c*) punching shear or by considering the depth required to prevent the pillar forcing its way through the foundation slab.

Fig. 515.

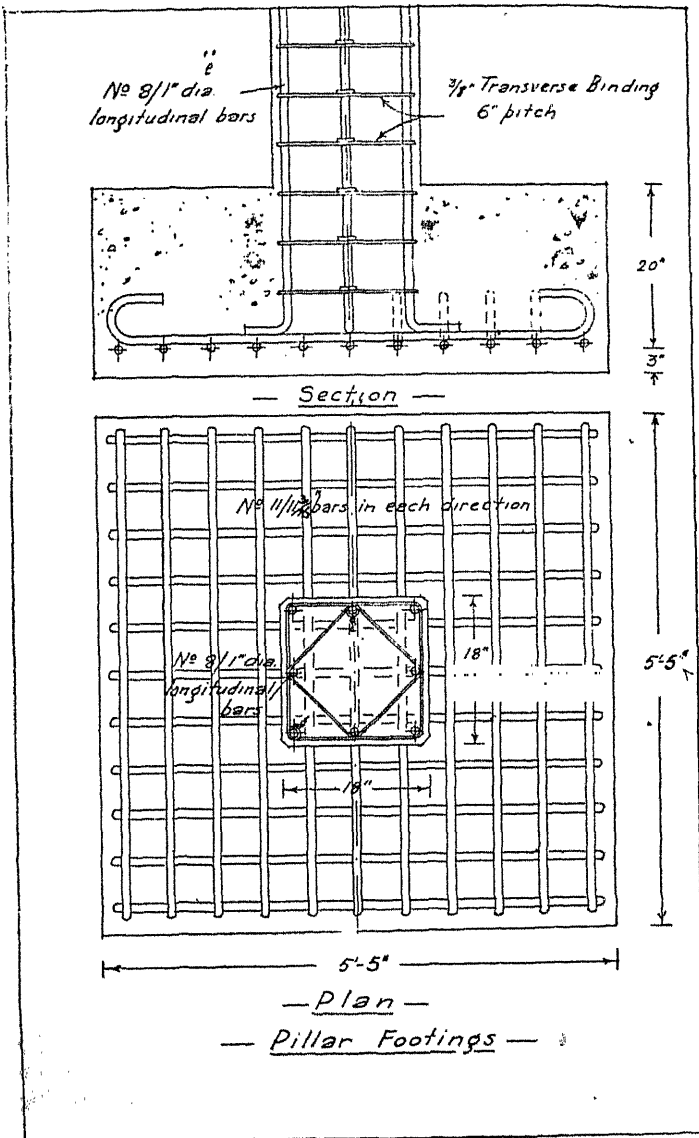


Fig. 516.

Figs. 515—516.

Consider the foundations for the external pillars (Figs. 515 and 516). Note that design height and weight of pillar and pillar casing have been taken through to underside of foundation slab.

Total load for table	= 189250
Assume additional weight of foundation slab	= 7500
Total	= 196750 lbs.

The safe resistance of soil = 3 tons/square foot.

Area of base required $= \frac{88}{3} = 29$ square feet.

Size of base $= \sqrt{29} = 5{\cdot}4$, say 5′−5″ square.

Depth required for shear parallel to pillar sides.

Shear on one plane $= \frac{W}{2} = \frac{196750}{2}$

$= 98375$ lbs.

$d = \frac{S}{sb} = \frac{98375}{75 \times 65} = 20{\cdot}1$ inches, say 20 inches

Depth to satisfy bending.

$$M = \frac{W}{8}\left(L - l\right) = \frac{196750\,(65 - 18)}{8} = 1156000 \text{ lbs. ins.}$$

$$d = \sqrt{\frac{M}{Qb}} = \sqrt{\frac{1156000}{137{\cdot}6 \times 65}} = 11{\cdot}4 \text{ inches}$$

∴ depth would be determined by shear.

Then $A_t = rbd = {\cdot}00893 \times 65 \times 20$

$= 11{\cdot}6$ square inches

Use No. 11/1 $\frac{3}{16}$-inch dia. bars in each direction.

Punching shear. Steel at 13000 lbs./inch.^{2}

Resistance of concrete	$4 \times 18 \times 20 \times 75$	= 108000 lbs.
,, steel	$12 \times 1{\cdot}1075 \times 13000$	= 172000 lbs.
		280000 lbs.

This is in excess of the requirements.

NOTE.—The punching shear has been taken vertically, but the area would be much greater due to the dispersion angles.

Internal Pillars.

The total load = 333400 (See Table.)

Assumed additional weight of slab = 17300

Total = 350700 lbs. = 157 tons.

Area of base.

$$A = \frac{P}{p} = \frac{157}{3} = 52 \text{ square feet.}$$

Use a slab 7′—3″ square.

Depth for Shear.

$$\text{Shear} = \frac{W}{2} = 175350 \text{ lbs.}$$

$$d = \frac{S}{sb} = \frac{175350}{75 \times 87} = 27 \text{ inches}$$

Depth for Bending.

$$M = \frac{W}{8}\left(L - l\right) = \frac{175350 \times 67}{8} = 1470000 \text{ lbs. inches}$$

$$d = \sqrt{\frac{M}{Qb}} = \sqrt{\frac{1470000}{137{\cdot}6 \times 87}} = 11{\cdot}1 \text{ inches.}$$

Therefore the depth is determined by shear.

Then $A_t = rbd = {\cdot}00893 \times 87 \times 20$

$= 15{\cdot}5$ square inches.

Use No. 14/1⅛-inch dia. bars in each direction.

Punching shear.

Resistance of concrete = $4 \times 20 \times 27 \times 75$ = 162000

,, steel = $16 \times 1{\cdot}1075 \times 13000$ = 230000

392000

This is in excess of the requirements.

Weights and Areas of Round and Square Bar Steel.

Diameter or side in inches.	Round bar.			Square bar.	
	Weight in lbs. per ft.	Area in square inches.	Circumference in square inches.	Weight in lbs. per ft.	Area in square inches.
3/16	·094	·0276	·589	·119	·0352
1/4	·167	·0491	·7854	·212	·0625
5/16	·261	·0767	·9817	·333	·0977
3/8	·375	·1104	1·1781	·478	·1406
7/16	·511	·1503	1·3744	·651	·1914
1/2	·667	·1963	1·5708	·850	·2500
9/16	·845	·2485	1·7671	1·076	·3164
5/8	1·043	·3068	1·9635	1·328	·3906
11/16	1·262	·3712	2·1598	1·608	·4727
3/4	1·502	·4418	2·3562	1·913	·5625
13/16	1·763	·5185	2·5525	2·245	·6602
7/8	2·044	·6013	2·7489	2·603	·7656
15/16	2·347	·6903	2·9452	2·989	·8789
1	2·67	·7854	3·1416	3·4	1·000
1 1/16	3·014	·8866	3·3379	3·838	1·1289
1 1/8	3·379	·9940	3·5343	4·303	1·2656
1 3/16	3·766	1·1075	3·7306	4·795	1·4102
1 1/4	4·173	1·2272	3·927	5·312	1·5625
1 5/16	4·6	1·353	4·1233	5·857	1·7227
1 3/8	5·049	1·4849	4·3197	6·428	1·8906
1 7/16	5·518	1·623	4·5160	7·026	2·0664
1 1/2	6·008	1·7671	4·7124	7·65	2·25
1 9/16	6·52	1·9175	4·9087	8·301	2·4414
1 5/8	7·051	2·0739	5·1051	8·978	2·6406
1 11/16	7·604	2·2365	5·3014	9·682	2·8477
1 3/4	8·178	2·4053	5·4978	10·41	3·0625
1 13/16	8·773	2·5802	5·6941	11·17	3·2852
1 7/8	9·388	2·7612	5·8905	11·95	3·5156
1 15/16	10·02	2·9483	6·0868	12·76	3·7539
2	10·68	3·1416	6·2832	13·6	4·000

The slab has been taken as uniform in thickness throughout. They are sometimes made pyramidal in form, for economy in the mass of the concrete. Unless the slabs are very large the saving is problematical as the cost of the formwork to form the pyramid and the extra

labour in depositing and tamping the concrete must be set off against the saving in concrete.

Hoop Steel for Stirrups.—Is obtainable in gauges 1 to 26, and in widths from $\frac{3}{8}$ inch upwards varying by $\frac{1}{8}$ inch.

Also from $\frac{1}{32}$ inch to $\frac{5}{16}$ inch in thicknesses varying by $\frac{1}{64}$ inch.

CHAPTER XXII

ROOFS

Continued from Elementary Course

Roof Trusses.—The methods of calculating the magnitude of the stresses in roofs have been given in the chapter on Statics, and the construction of roofs has been dealt with somewhat at length in the *Elementary Course*, and the following will only be mentioned here: Mediæval, and curved rib trusses.

Mediæval Trusses.—It may be convenient to trace the development of the trusses used in the mediæval roofs under five headings. The principle throughout is to use timber to resist compression and transverse stresses, and to reduce the tensional members to a minimum.

1. Roof trusses, the outline of which was similar to the couple-close truss (Fig. 517) and the horizontal member of which was supported along its entire length by masonry or brickwork in its turn, supported by a stone arch, the abutments of which were the main walls; and having one central, vertical member in the truss under compression, this member being known as a crown-post, were known as tie beam roofs.

2. A truss of the outline of the collar-beam truss (Fig. 518) was placed instead of a couple-close truss, all other parts being similar to the first. This made it possible for the arch in a similar sized building to be of greater height.

3. A timber arch was substituted for the stone arch and wall under the truss of the first type in the form of a pair of brackets, the curved struts and horizontal members performing the work of the arch and wall, as shown in

Fig. 517. This construction is known as the arched tie-beam. The strut members of the brackets were sometimes put in straight.

4. Later, a wood rib was substituted for the stone arch in the collar beam type, as shown in Fig. 520. Instead of the wood rib being under compressional stress

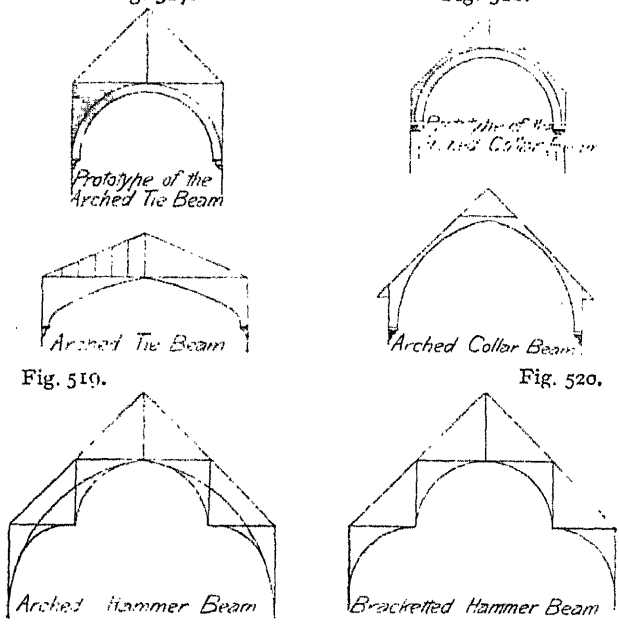
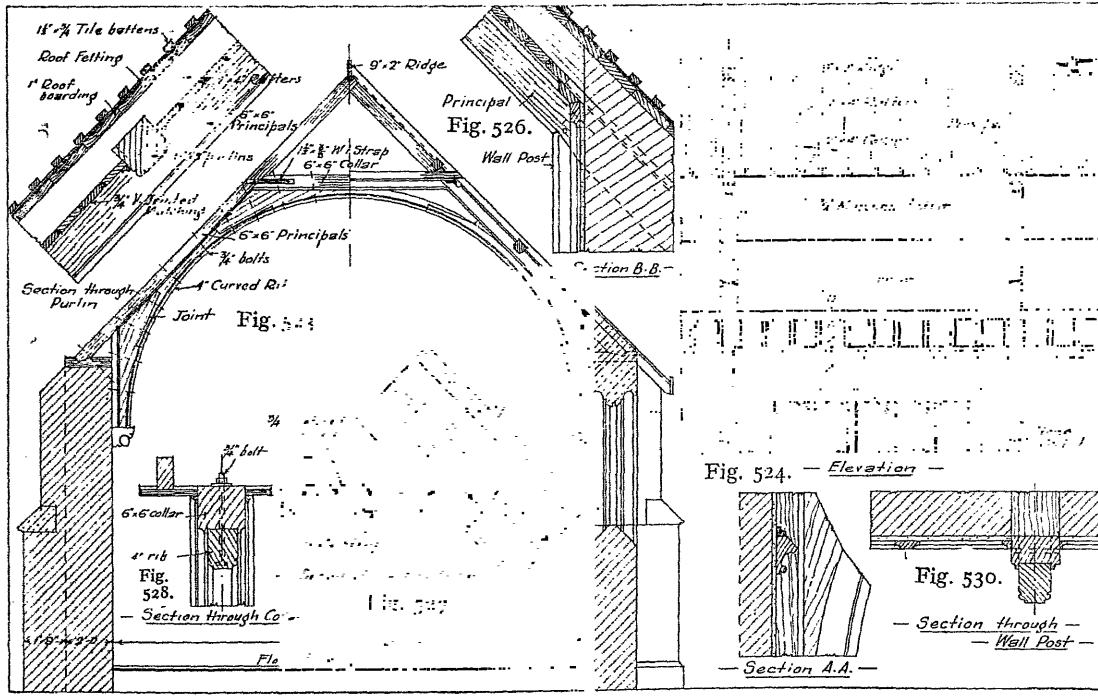


Fig. 517. Fig. 518. Fig. 519. Fig. 520.

Fig. 521. Fig. 522.

Figs. 517—522.

only, it is here also under transverse stress, being used as a gusset piece to prevent the deformation of the joint between collar and principal and wall post and principal, and at the same time to transmit the compressional stress at a great distance below the top of the wall, as shown in Figs. 523 to 530. This form of truss does not detract from the height of the chamber, the cover-


1½"×¾" Tile battens
Roof Felting
1" Roof boarding
Principals
Purlins
¾" V Jointed Boarding
Section through Purlin
Fig. 525.
9"×2" Ridge
Principal
Fig. 526.
Wall Post
Section B.B.
6"×6" Principals
¾" bolts
1" Curved Rib
Joint
Fig. 527.
Fig. 524. — Elevation —
¾" bolt
6"×6" collar
1" rib
Fig. 528.
— Section through Co —
Fig. 530.
— Section through — — Wall Post —
— Section A.A. —
Fig. 529.

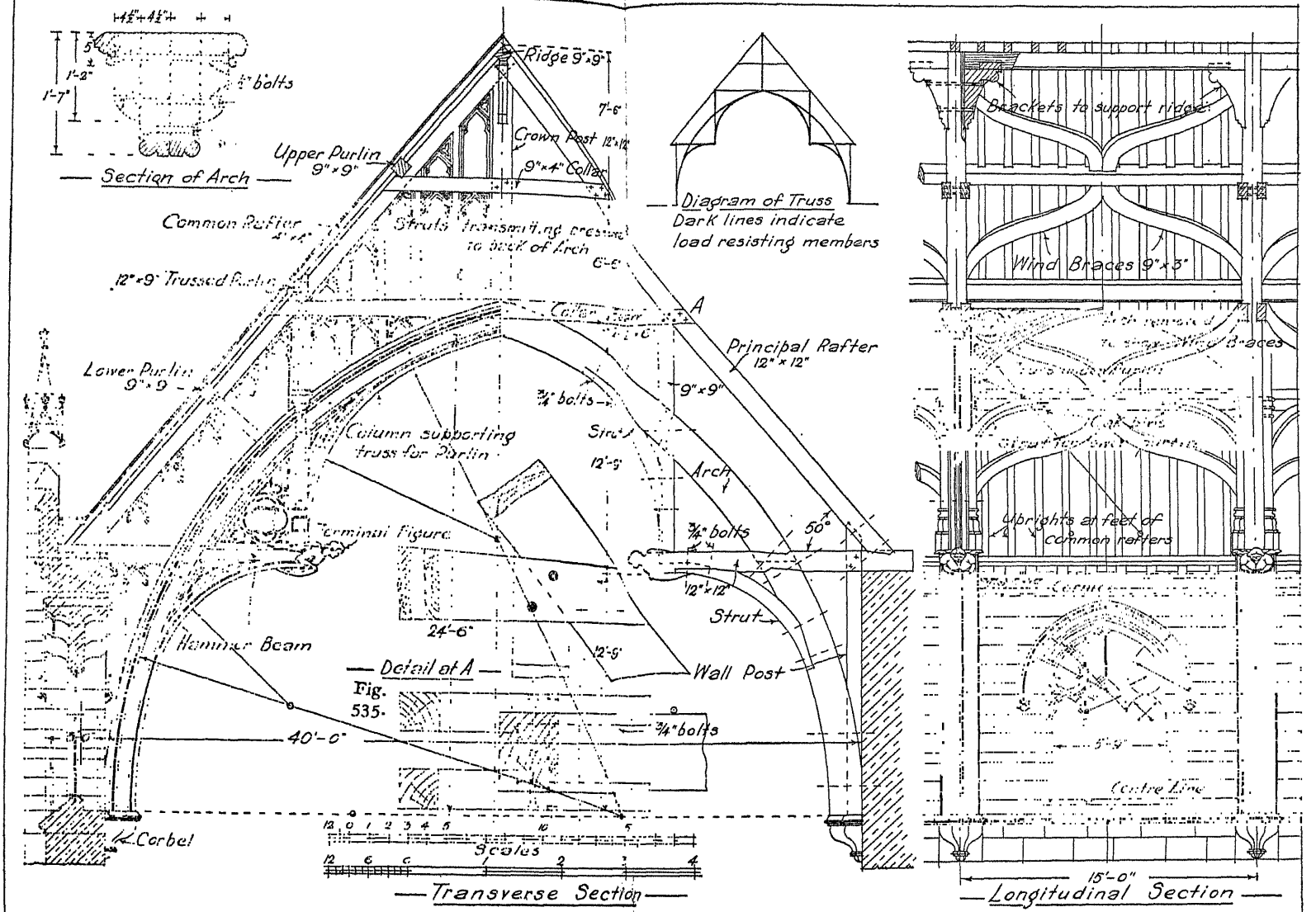
Figs 523—530.

ing of which it supports, and is suitable for spans of about 20 feet.

5. For larger spans than 20 feet the hammer beam principle is preferable. This is of two kinds, the arched and the bracketed, as shown in Figs. 521 and 522.

(*a*) Figs. 531 to 535 show an example of the arched type. In this the main constructional member is the timber arch. This is formed from four or more pieces bolted together and moulded. The crowning moulding of the arch is simply an ornamental member. The tendency of an arch when loaded at the crown is to sink at the latter part and for the haunches to rise. In this type it is prevented by the insertion of a horizontal member known as a hammer beam projecting at the level of the foot of the principal rafter, to which the latter is framed, and an upright at the extremity of the hammer beam which triangulates the arch at that point ; this tends to prevent the above-mentioned deformation. This combination is assisted by two struts, which further extends the influence of the former. The struts are usually curved, so that they may flow into the curve of the large arch, and thus prevent the appearance of anything approaching a broken or crippled curve, and giving the appearance of a cusped arch. The weight of the rafters and roof coverings is transmitted to the principal rafters by means of the purlins, the principal rafters transmit the weight to the arch through a number of vertical studs, the latter being tenoned to the principal rafters and the back of the arch. The ridge in this type usually consists of a heavy beam supporting the common rafters as shown. This is supported on brackets secured to uprights termed crown posts, which transmits the weight to the centre of the arch. The spaces between these upright members are usually fitted with small cusped frames for decorative effect. The lowest purlins are stiffened by means of two struts usually curved, as shown in Fig. 533, the foot of the struts resting upon the back of the arch. The centre purlins are stiffened by means of a vertical wood arch, which spans the space between the two trusses and springs either from the back of the main arches or from a small column, as shown in Figs. 532 to 535, secured to the upright that rests upon the extremity of the hammer beam. This

Fig. 534. Fig. 532. Fig. 531. Fig. 533.



Figs. 531—535.

[*Between pages 792 and 793*

arch has its spandril invariably filled with tracery, which gives to the whole composition a very rich effect. The roof is stiffened longitudinally by means of members known as wind braces, which are placed against the underside of the rafters, and abut upon the principals. These are usually curved for effect.

The arch rests upon a stone corbel or the capital of a wall column at some distance below the top of the wall.

The increased unobstructed height of the chamber, obtained by trussing in this manner, adds greatly to the internal majestic appearance of the roof, but the rigidity of the framing depends to a large extent upon the immovability of the abutments.

(*b*) At a later period a similar type of roof to the above, omitting the arch, as shown in Fig. 522, was used. This alters the principle of the truss entirely, which now practically becomes two braced girders, tilted and butting against each other in the centre. The curved members of the brackets are now in tension, and their function could be better performed if they were straight. In other respects the constructional details are similar to the preceding example shown in Figs. 531 to 535.

Flèche for Ventilating.—Figs. 536 to 541 show a flèche for ventilating purposes such as would be in conformity with the mediæval roofs previously described. These flèches may be in plan hexagonal or octagonal. Being subject to considerable wind pressure, it is necessary to frame them well down into the roof. Figs. 536 and 538 show an example fixed down to the collar beams of a roof. The flèche is placed centrally between two trusses, the collars of the latter being connected by two cross beams, and these again have two transverse members framed into them, forming a square that encloses the timbers placed at an angle of 45 degrees which form the octagonal base of the flèche. The faces of the octagonal pyramid are inclined at an angle which varies between 80 and 85 degrees. The hips are birdsmouthed into the angles of the octagonal base, and at their upper ends are nailed to a central mast. A series of purlins are placed at the sill level; and to form a base for the mast a series of ties, constructed as shown in

Fig. 541, are fixed to the hips, the masts being joggled into this series of ties. The part of the structure below the sill is braced, as shown in Figs. 536 and 538. The buttresses are framed and bolted to the hips, the louvre frames are constructed as shown in Fig. 539, tongued into the buttresses, the whole of the sills being housed into the latter. The upper portion of the flèche is then boarded, the gables and the rolls being formed as shown in Fig. 537; it is then covered in with milled lead. The lead covering extends over the crowning moulding of the gable. The pinnacles are of cast lead fixed over the wood cores forming buttresses. The crockets on rolls and gables are of cast lead. The sheet lead covering is usually laid to the herring-bone pattern as shown, and welted at joints. The louvres below are formed of stout copper or zinc turned up at its back edge. The rain-water from the roof is drained away through a lead pipe forming a gargoyle, the lead pipe passing through the buttress, as shown in Fig. 538.

Curved Rib Trusses.—The maximum interior height of buildings has been attained by using curved rib trusses. These may be made in timber in two ways: (*a*) built-up curved vertical plates; (*b*) built-up ribs with thin plates bent to the contour of the curve. In the latter case the plates are bonded by the built-up method.

Belfast Truss.—(Figs. 543–545) is a cheaply formed truss much used for factory buildings and temporary work. It is built on the principle of the arch or the bowstring girder. The stresses at any part of the curved rib and the tie is determined as follows:

The load consisting of the purlins and coverings may be considered as uniformly distributed over the truss, and the bending moment equals $\frac{Wl}{8}$. The horizontal tension in the tie H_T equals Hd, and equating these

$$\frac{Wl}{8} = H_T d$$

$$H_T = \frac{Wl}{8d} \quad \ldots\ldots\ldots\ldots \quad (1)$$

Fig. 536. Fig. 540. Fig. 537.

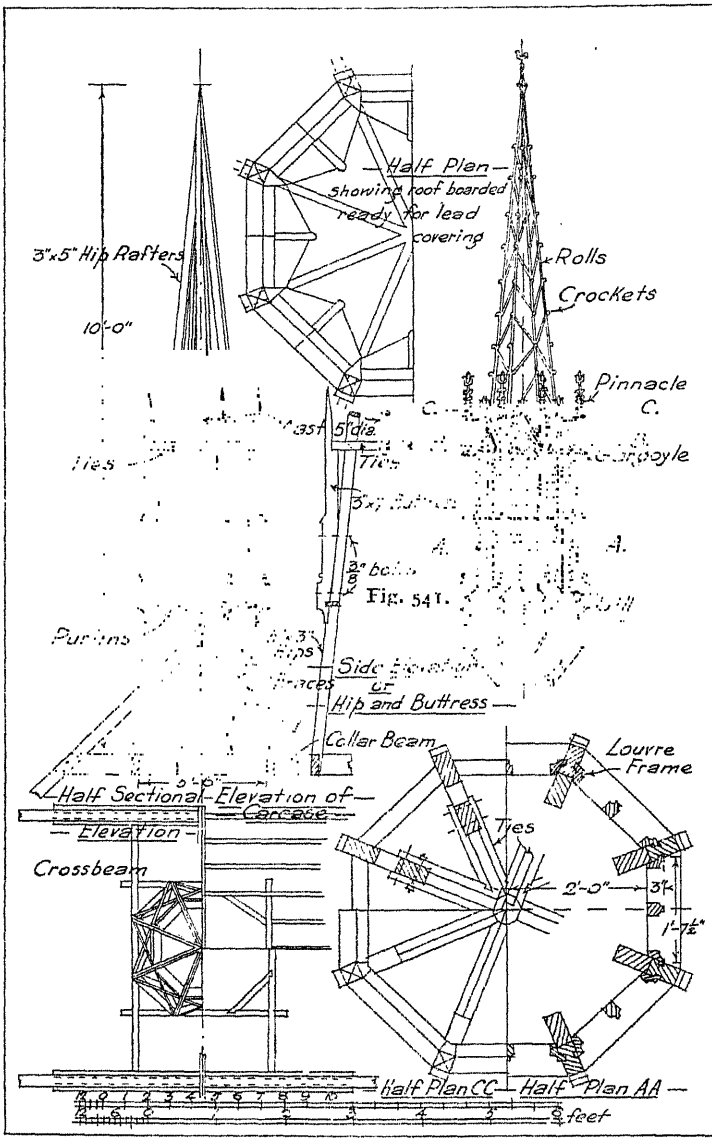


Fig. 538.

Fig. 539.

Figs. 536—541.

This value of H_T is constant at any point in the bow, and the thrust T in the bow at any point is H_T resolved into the angle made by the tangent to the bow at that point and the value of $T = H \sec \theta$ . . . . . (2)

The curve of the bending moment would be a parabola. This is an inconvenient curve for general work, but if the curve of the rib be made a segment of a circle, with a rise not exceeding one eighth of the span, it will very closely approximate to the curve of a parabola having that altitude. If the direction of the load was vertical there would thus be no deformation of the rib under stress, but variations caused by wind would tend to cause deformation.

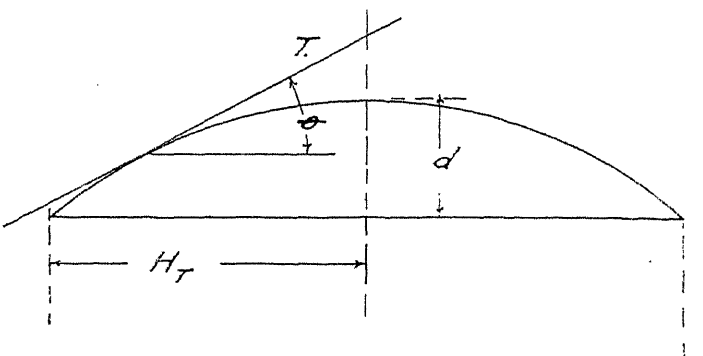


Fig. 542.

To prevent this, a lattice of light section bars is employed that connects the curved rib with the tie and confers rigidity on both members.

Trusses up to 100 feet span may be made on this principle.

The tie and ribs for one truss are made in two parts or pairs with the lattice between. The tie is formed of 9″ or 11″ × 3″ boards, in two or more thicknesses, having their joints staggered, the pieces being securely nailed together. By this means a bar of any desired length may be produced. The curved rib may be made in two or more laminæ, about 1½ inches in thickness, laminated and bent to the required curve, the joints, as in the tie, being staggered. The lattice bars are formed of small battens from

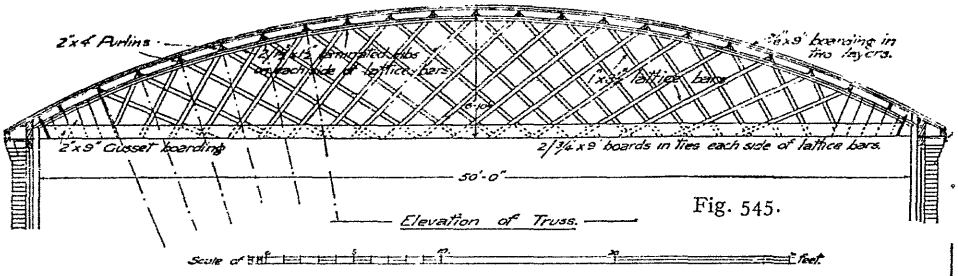


Fig. 545.

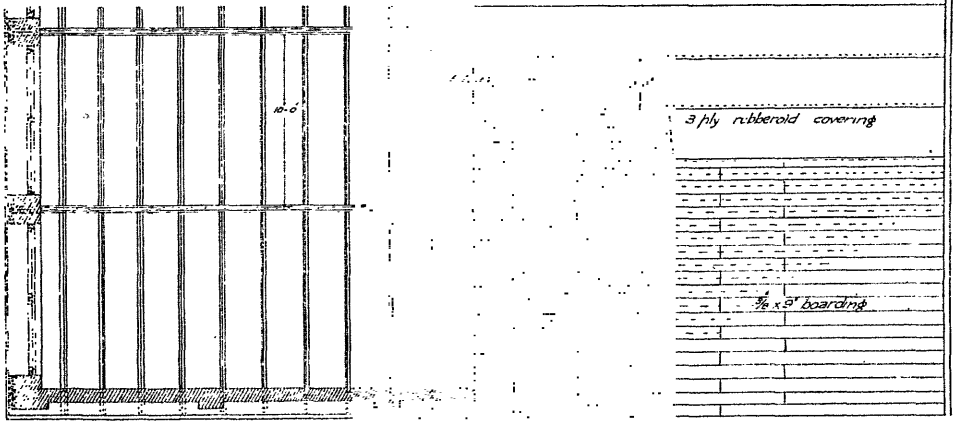


Fig. 543. Fig. 544.

Figs. 543—545.

2″ × 1″ upwards, according to the span of the roof. The lattice bars are placed in pairs at right angles to each other, the axial line between them being placed normal to the curve; they project beyond the curved rib sufficient to form a seating for the purlins, as shown in Fig. 545. The purlins are made from 2″ × 4″ or 2″ × 5″ according to span, and at intervals of not more than 2 ft. 6 in. apart. The covering is made from boarding not more than ¾ inch in thickness, preferably in two layers, ⅜ inch in thickness, laminated and laid with their longitudinal joints broken. These can be easily bent about the curve, and when nailed to the purlins make with the rib a very strong arched surface.

The curved ribs are bridled into the tie at their lower extremities, and secured with iron straps. The ends of the truss which are subjected to the greatest shear are further strengthened by boarding 9 inches × the combined thickness of the two lattice bars. An alternative to this is to make the lattice bars near the ends of an increased width. The whole of the joints of the truss are secured by nailing, including the crossings of the lattice bars. The roof surface is covered with Ruberoid or some similar flexible covering, or by corrugated iron bent to the requisite curve. The whole forms a cheap, rigid truss suitable for factory work or for works of a temporary character. (For details, see Figs. 543 to 545.)

CHAPTER XXIII

JOINERY

Continued From the Elementary Course

In considering the following examples reference should be made to the general principles laid down for the design of joinery in the *Elementary Course*, also to the remarks on Seasoning in Chapter on Timber (p. 178). Attention is particularly directed to the inherent tendency in wood of all kinds to react to the variations of the humidity of the atmosphere, and in designing woodwork for a particular position to have regard to the possible moisture content of the wood in that situation, and to make provision for the possible movements of the material.

Flooring.—Flooring may be classified as :—

(1) Batten.
(2) Block or Parquetry.

Floor boards are laid by one of the methods shown in the *Elementary Course*. The floor boards in the best floors at the present time are generally laid with the lateral joints of the section, as shown in Figs. 646 and 656 of the above work.

It is imperative where the battens are out of hard wood, and are intended to be polished, that every precaution be taken to prevent the joints opening from shrinkage. This is accomplished by using narrow battens. If of hard wood these should not exceed 3 inches in width, be thoroughly seasoned, and the joints glued with a thin compound of glue and whiting.

Polished wood floors are usually laid upon a counter floor.

Counter Floors.—These consist of ordinary battens usually with a grooved and tongued joint, laid diagonally upon the joists in order that none of the joints shall coincide with the joints in the floor above. The counter floor, which is usually prepared from pine or spruce, should be carefully cleaned off to a plane surface. This is best done with a mechanical sander. Upon this the hard wood battens, which have been carefully thicknessed, are fixed through the edges of the boards; these should also be cleaned off with the sander, to avoid any plane marks.

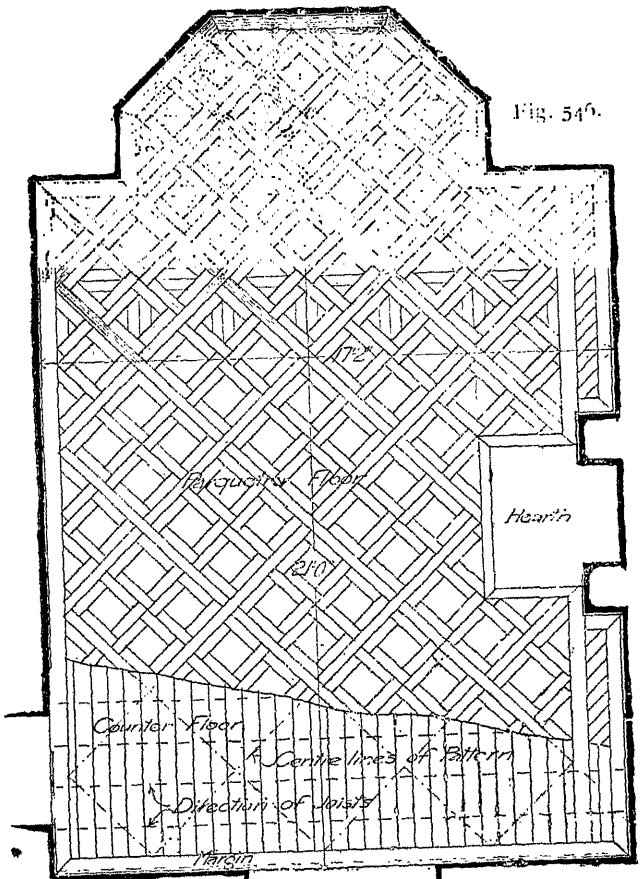
Parquetry Floors.—Counter floors instead of being covered with battens are often overlaid with small blocks about 1 inch in thickness, laid to various patterns either dowelled or tongued together, and glued both to the counter floor, and to each other as shown in Figs. 546 to 548. Where the pattern lends itself to be so treated, the blocks are glued up in squares varying from 12 to 18 inches length of side before being fixed, for facility in laying.

The joints between the squares are grooved, tongued, and glued, the tongue being placed two-thirds of the thickness from the top, and fixed on two of its edges with a screw.

Where patterns similar to the above are used, a straight border is usually fixed about the room to be treated, wide enough to extend beyond all small projections in the skirting, such as the bases of pilasters, etc., in order to form a straight line against which the pattern may be finished. The border is returned about all large projections, chimney breasts, etc. The distances between the borders should be set out to avoid all irregular cutting of the patterns, multiples of the latter being preferable.

Plain Wood Blocks.—These are often laid to herring-bone and other patterns, and are fixed by bedding in a mastic cement, composed in the proportion of 1 cwt. pitch to $7\frac{1}{2}$ gallons of coal tar, boiled together for at least one hour, so that when cold it may be elastic or tough if laid on concrete; or a compound of glue and whiting if laid on a counter floor. The long bottom edges of the blocks have a dovetailed shaped groove taken out of them

Fig. 546.



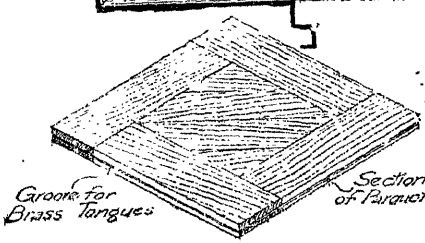


Fig. 547.

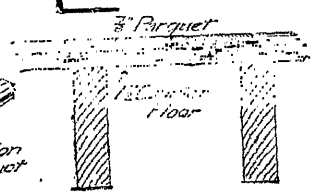


Fig. 548.

Figs. 546—548.

in order to key the block to the cement, and at the ends there is a small dovetailed shaped projection to key one block to the other.

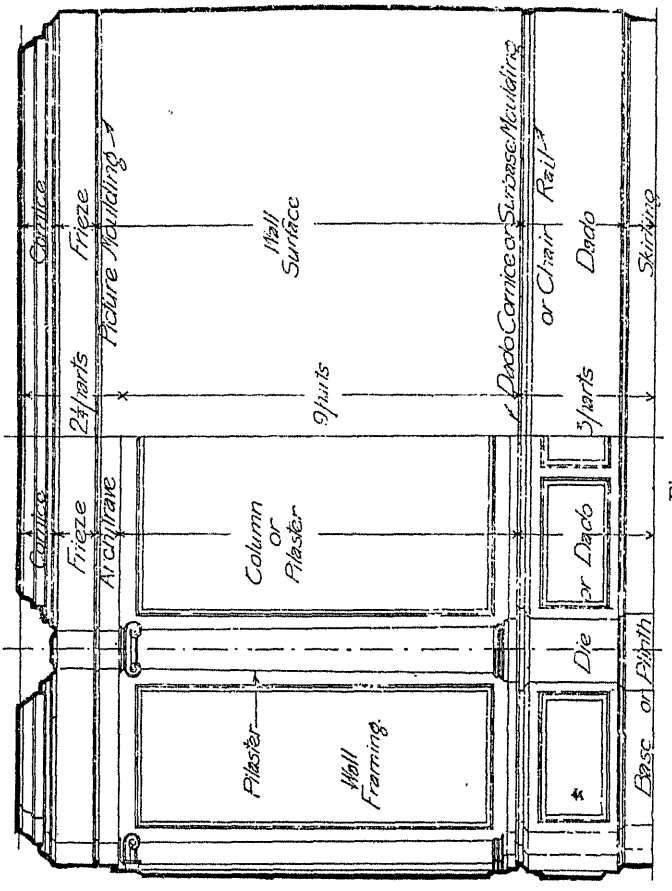


Fig. 549.

Wood Finishings.—The wood finishings of a room often or partly correspond to the parts of an order in the Roman Classic, the members of the latter including plinth, die,

cornice of die, column or pilaster, architrave, frieze and cornice, The above details are usually known as skirtings, dado, surbase mouldings or chair rails, wall framing, architrave moulding or picture rail, frieze and cornice as shown in Fig. 549. The absence of wall framing and columns or pilasters in rooms should not interfere with the proportions of the skirting, chair rail, picture rail, width of frieze or cornice.

Although decoration by means of joinery work, based on Classic traditions, will probably continue, the introduction of new materials and new applications of materials that have long been in use, in addition to hygienic considerations and the necessity for labour saving, has resulted in a new conception of joinery finishings based principally on utilitarian requirements, and the elimination of everything that serves no definite purpose. Probably this tendency has been more active in the joinery and furniture trades than in any other department of building. The introduction of plywood and lamina boards in their various forms, has made it possible to cover large surfaces without the necessity of framing. Also the process of veneering the surfaces with veneer cut from many of the beautifully figured woods that have been recently introduced from many parts of the Empire has developed methods of decoration that have no direct connection with traditional methods.

Skirtings and Dados.—Skirtings in joinery consist of boards fixed about the lowest part of the walls of rooms, primarily to protect the wall from damage and secondly to form a decorative finish.

To fix any joinery to walls so that the lines shall be straight and the plane surfaces true, it is necessary first to fix a system of rough grounds. These consist of narrow bands of wood 2 inches wide by $\frac{7}{8}$ inch thick, the latter being the thickness of the plaster. The grounds should be placed behind the frame members and not the panels, so that the work may have a solid bearing. The grounds are nailed to either wood plugs, wood bricks, or special fixing blocks, or a course of $9'' \times 4\frac{1}{2}''$ breeze slabs are built in behind where the joinery members are designed to be fixed. Where two or more horizontal grounds occur in the same

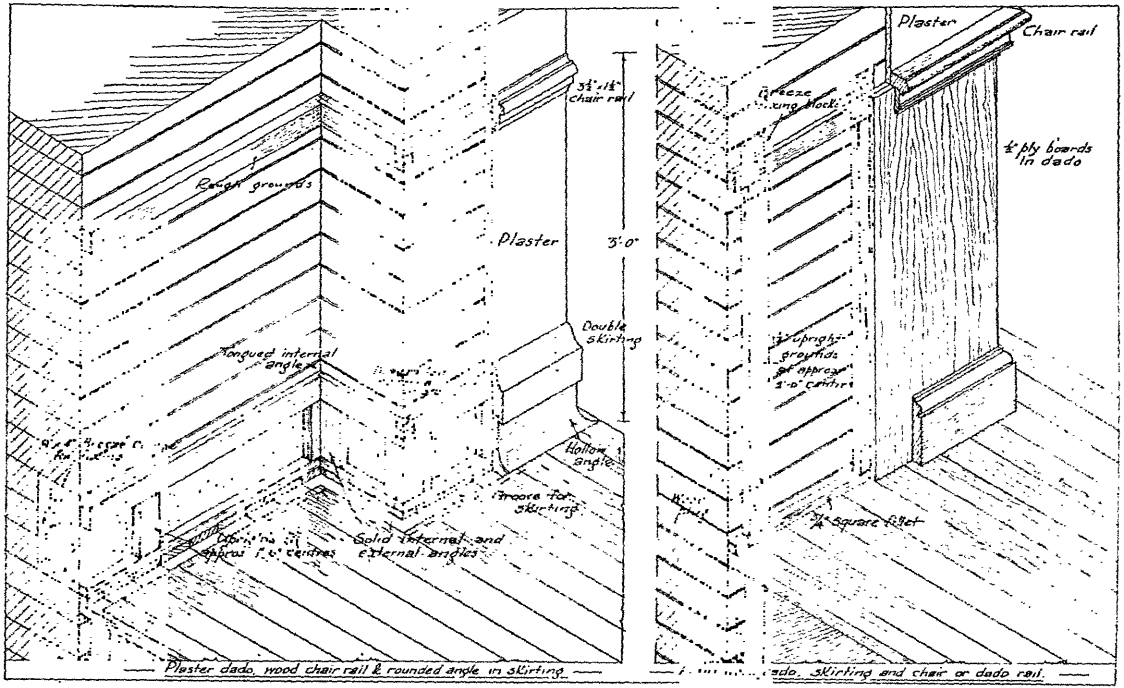


Fig. 550. Figs. 550—551. Fig. 551.

plane, they should be plumbed and should be tested for straightness with a long straight edge. All external angles about breaks should be dovetailed and internal angles tongued. See Fig. 550.

Fig. 550 shows the grounds and backings for the skirtings and dado rail on a wall with a plastered surface. The skirting ground is fixed with its upper edge half an inch below the top of the skirting, a $\frac{3}{4}$-inch square fillet is nailed to the floor boards, vertically below the ground, and upright blocks are fixed between the two at intervals of from 1 ft. 6 in. to 2 feet apart; the internal and external angles should be formed solid as shown in Fig. 550. The figure shows a double skirting, the lower portion of which is formed with a hollow of 1 inch radius to avoid any harbourage for dirt. Its lower edge must be sunk or tongued into the floor as shown. The upper portion is rebated over the lower. The chair or dado rail is fixed in a similar manner at the height of a chairback, to prevent injury to the wall from those articles of furniture. The skirtings are mitred at their external angles, and tongued and grooved on their plane surfaces, with the moulded portions scribed at the internal angles.

Fig. 551 shows a moulded skirting scribed to the floor in conjunction with a $\frac{1}{2}$-inch plywood dado. The plywood sheets can be obtained up to 4 feet in width. The grain of the outside ply is shown vertical, in this case it would require a vertical ground in the centre of the sheet, and the edges of the sheets would butt on a ground so that each sheet would be supported in three places. It would require a horizontal ground at the top and a $\frac{3}{4}$-inch square fillet nailed to the floor. Thus the sheet has a solid support along its four edges. The dados are here shown 3 feet in height, but they may, of course, be made any height.

Fig. 552 shows a double-faced skirting at the base of a framed dado. The lower part of the skirting is shown tongued to the floor and is fixed to the backings along its top edges; at its junction with the floor a triangular fillet is fixed in lieu of the hollow; this is less expensive. The figure 552 shows the arrangement of the grounds and block backings for the skirting, also a moulded dado rail.

Fig. 553 shows a heavy moulded, built-up skirting with

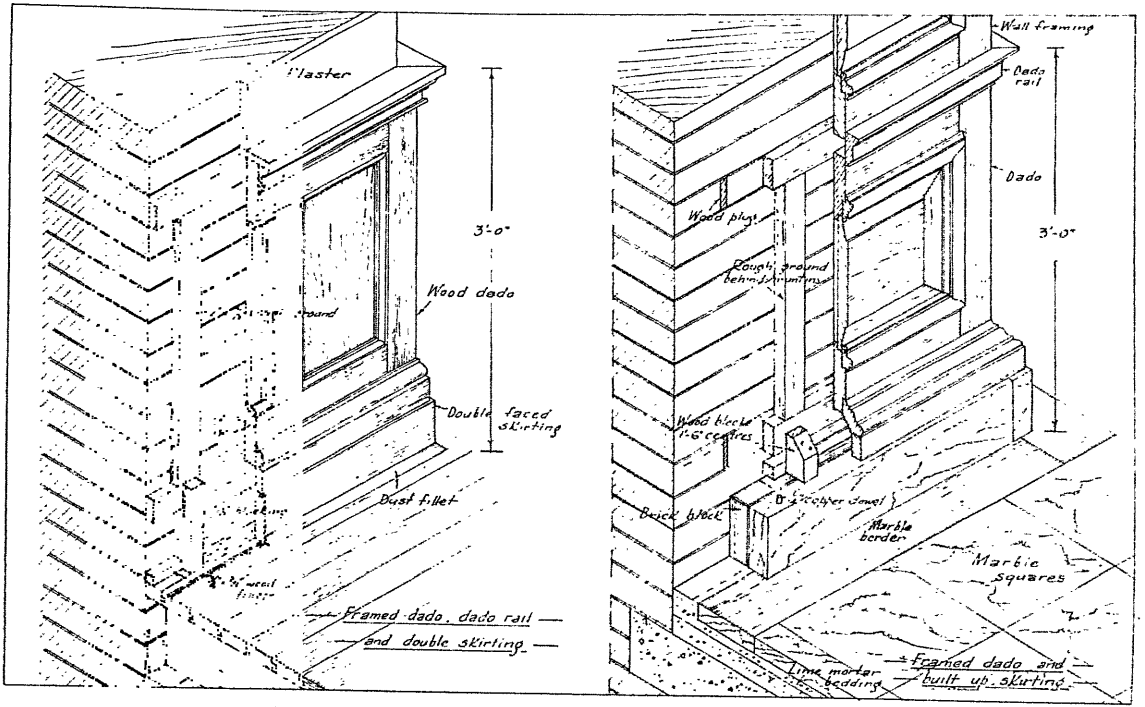


Fig. 552. Figs. 552—553. Fig. 553.

a framed dado and wainscot panelling above. The floor in this case is shown of marble squares; on this is a subskirting of marble. The squares should be bedded on lime or plaster; this is shown overlaying a low grade concrete, about 10 to 1, in which is embedded the electric conduits or other piping. The marble subskirting should be at least 2 inches thick, bedded on the floor and blocked out from the wall with brick or tile bedded in plaster. The marble blocks are dowelled together at the heading joints and cramped together at the angles. The figure shows the arrangement of the grounds and the blocking for the skirting. The lower member of the wood skirting is dowelled to the marble at intervals of about 1 ft. 6 in. The upper member is fixed to the blocking and, as it is finished with a hollow, it is rebated into the upper frame which would be fixed prior to the skirting.

Wall Framing.—Where walls are entirely covered with framing, the frames must be designed in sections of dimensions that can easily be transported and passed through doorways. Where large surfaces are covered with plain panelling without any natural breaks, the jointing of the sections will require careful consideration. Where the surfaces are broken up either by sunk or projecting pilasters the planning of the parts is much simplified.

Where the walls of rooms are to be entirely covered with framing, a plan to a scale of at least 1 inch to the foot should be set out, on which the arrangement of the grounds can be indicated and the dimensions of each section of the framing can be figured. This enables the maximum of the work to be done in the factories, and considerably facilitates the process of fixing, and this, in addition to reducing the cost, produces better work.

Figs 554 to 558 show an example of plane framing. The sections in this case would be too large, if put together, to get through the doorways. In this case the vertical members are continuous from top to bottom (see the joints in Fig. 558). This admits of preparing the work in narrow sections which can be connected when in the room.

Figs. 559 to 562 show a case with projecting pilasters. The intermediate panelled sections are usually sufficiently

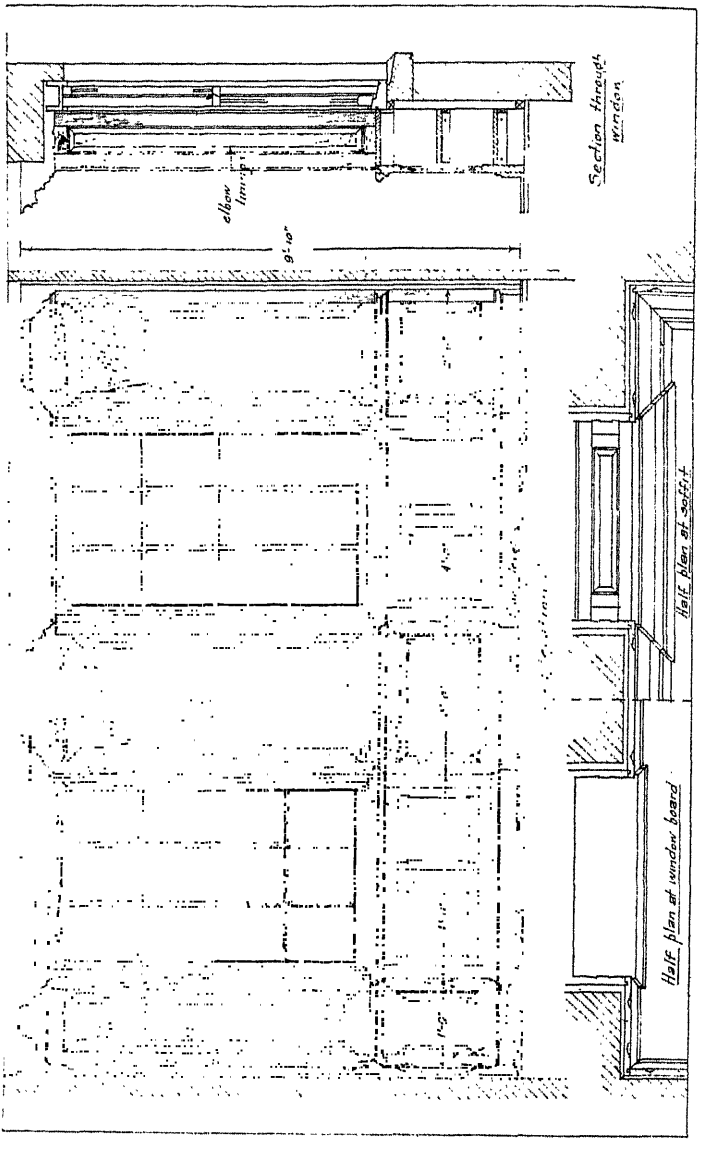


Fig. 554.

Fig. 555.

Fig. 556.

Figs. 554—556.

Fig. 558.

Fig. 557.
Figs. 557—558.

small to be passed through the doors, and the sections can be glued up. The pilasters, which can be fitted at the works, are laid over the edges of the panelled portions (see Fig. 561). Figs 564 to 573 illustrate a wall panelled with sunk pilasters. The latter are in narrow widths and are thinner than the main panelled sections. They are tongued to the larger frames (see Figs. 564 to 568 for the general views, and Figs. 569 to 573 for large details of the parts).

The panels in much of the seventeenth and eighteenth century period work are sometimes filled with tapestry or fabric, and in other cases some of the panels are fitted with mirrors. Framing of this type must be fitted with great accuracy, especially in the latter case, as any want of uprightness or parallelism will cause the reflexions of the opposite surfaces to appear out of plumb and irregular.

Gothic Framing.—The tracery in the Gothic styles is usually fret-cut and glued upon the panel with the grain in the same direction as the panel, as shown in Fig. 563.

The mouldings in framing of the Gothic styles are usually stuck on the solid. The mouldings are not jointed at the angles in the usual way, but are returned about the angles by what is known as a mason's mitre, as shown in Fig. 563. The framing is first fitted together; the mouldings are invariably stuck on the full length of the muntins, and the rails and styles are stopped where the muntins and rails respectively abut; the framing is then glued up, and held together by pins driven through the face of the frame, and passing through the tenon. The mitres are then carved at the angles. The upper horizontal edges of the members are often chamfered, not moulded (as shown in Fig. 563), the moulding on the vertical members either dying into the chamfer or being stopped a few inches above the chamfered edge.

Figs. 574 to 581 illustrate the application of Gothic framing to a bench end. Each bench, forming one of a series, usually has the framing tenoned into a horizontal plate extending the whole length of the system. The panel is prepared with the tracery upon it, and is inserted between the styles, which are then connected at their upper ends by

Fig. 560.

Fig. 562.

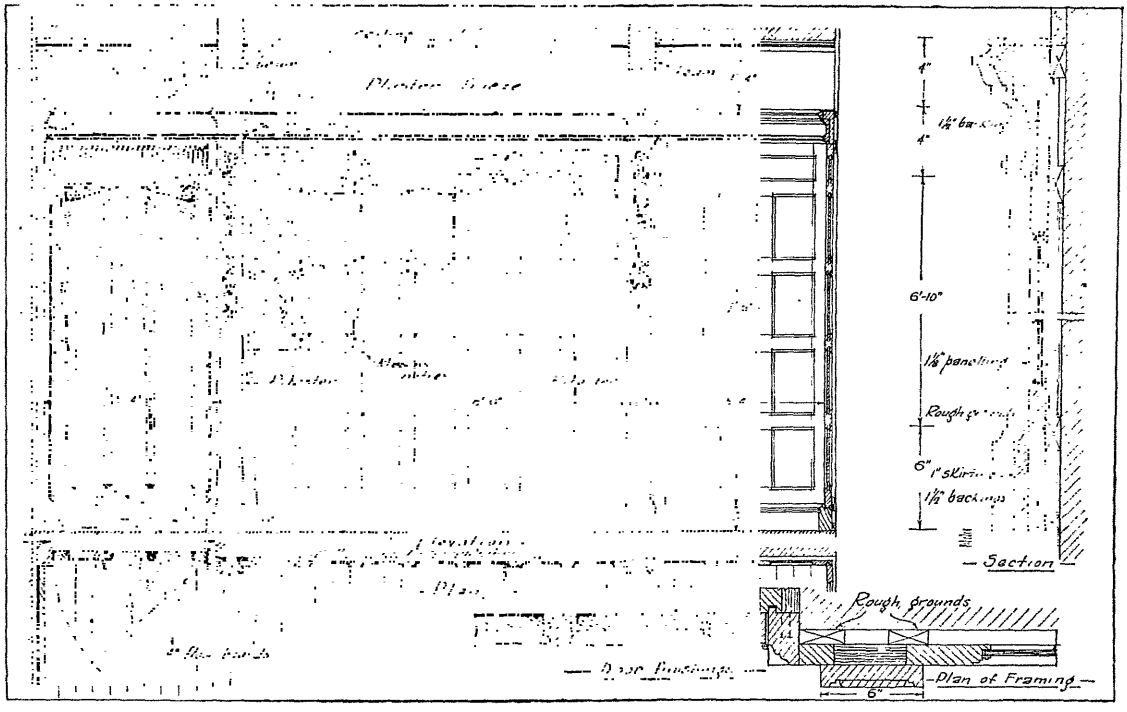


Fig. 559.

Fig. 561.

Figs. 559—562.

the top rail, which is mortised to receive the tenons on the styles. This is an exception to the ordinary method of framing in which the styles are mortised. Buttresses are here placed on the outside of the styles, and although their function is decorative, they also serve to greatly stiffen the

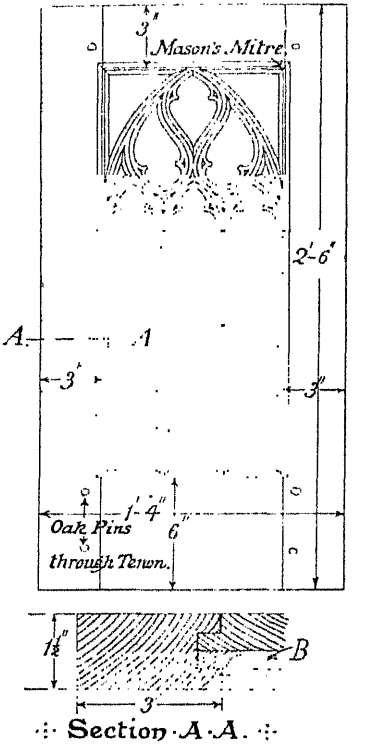
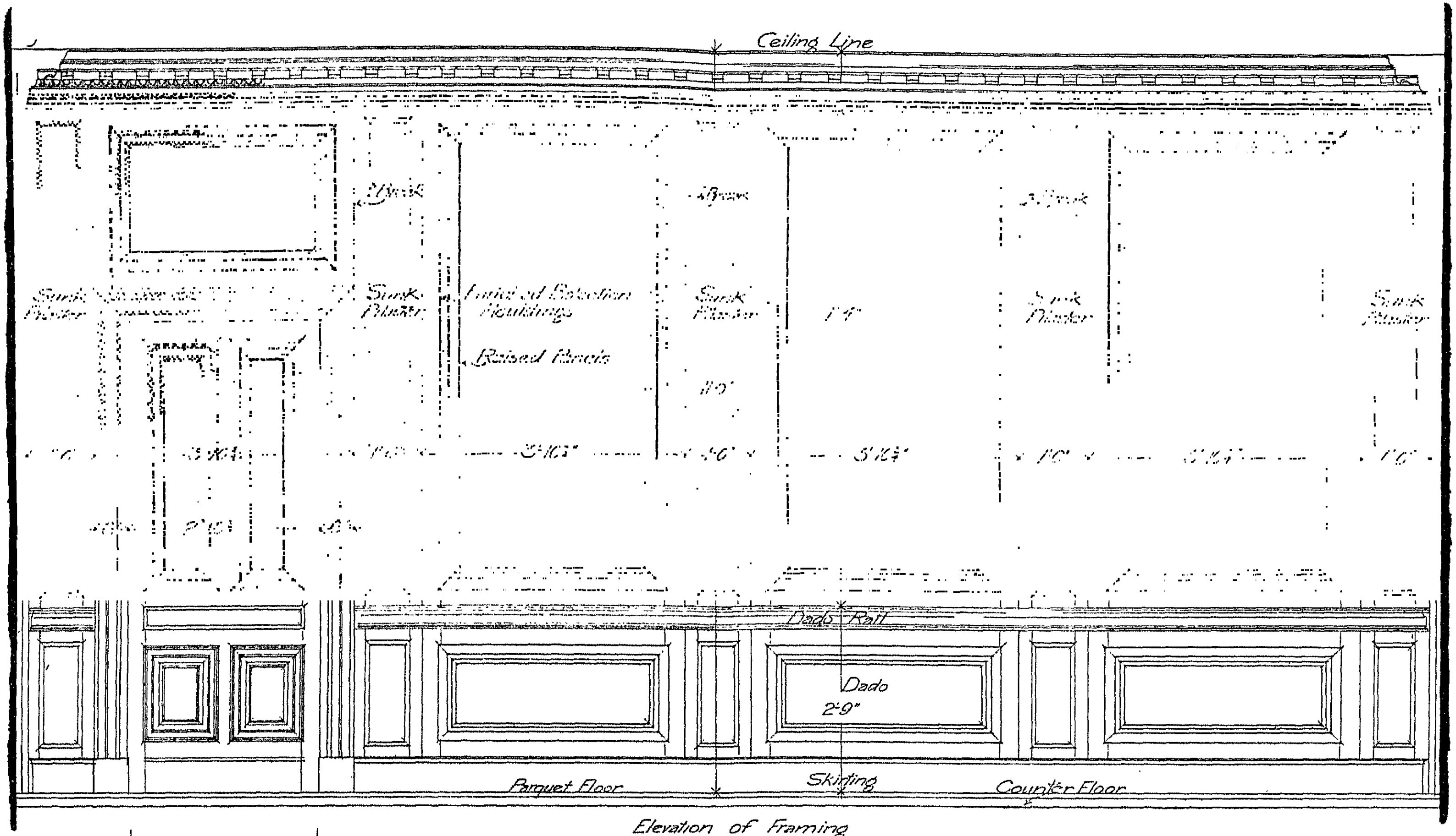


Fig. 563.

frame. Small joists about 2 inches in thickness are placed about 1 foot apart between the horizontal plates, and the floor boards are fixed. The seat backs consist of panelled frames, the panels usually being of V-jointed matching. They should be panelled to the ground to completely separate each pew from those adjoining, and the frame

Fig. 564.



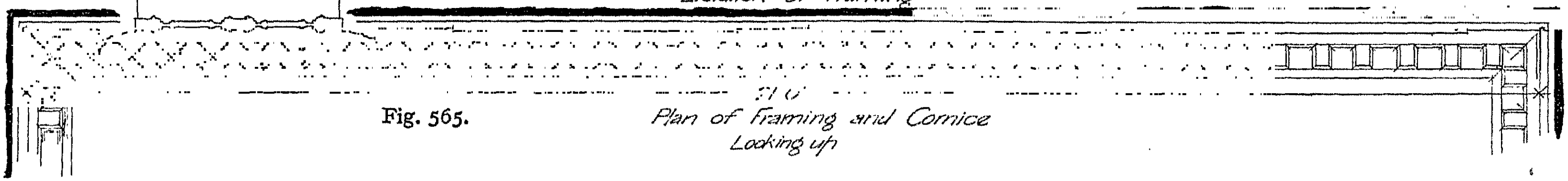


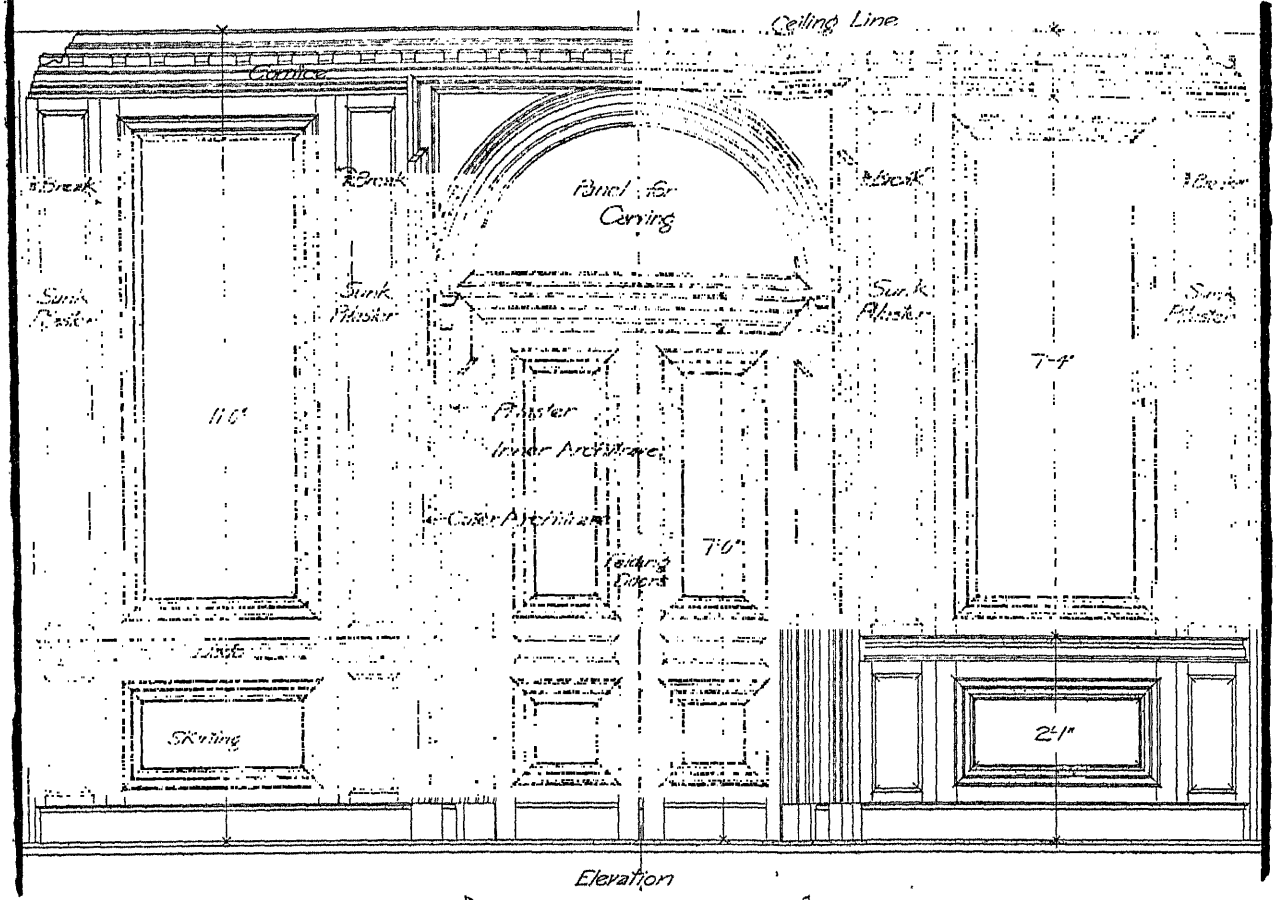
Fig. 565.

Plan of Framing and Cornice
Looking up

Figs. 564—565

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Fig. 566.



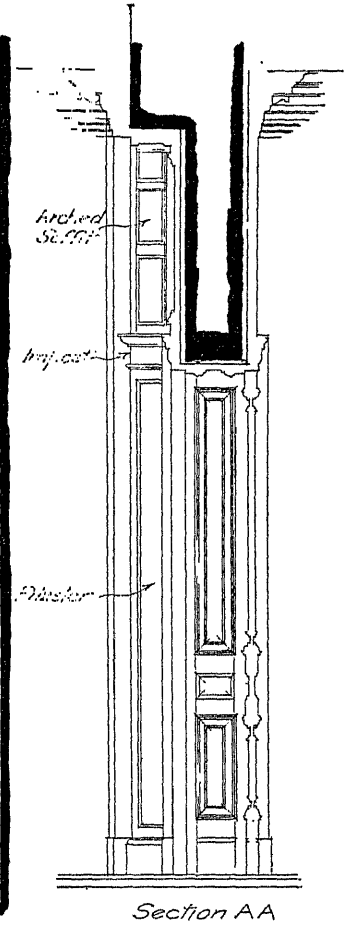
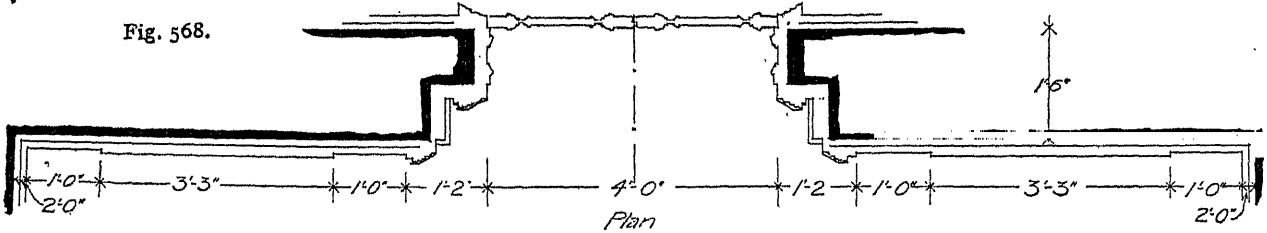


Fig. 567.

Fig. 568.



Figs. 566—568.

[*Between pages* 812 *and* 813.

Fig. 569.

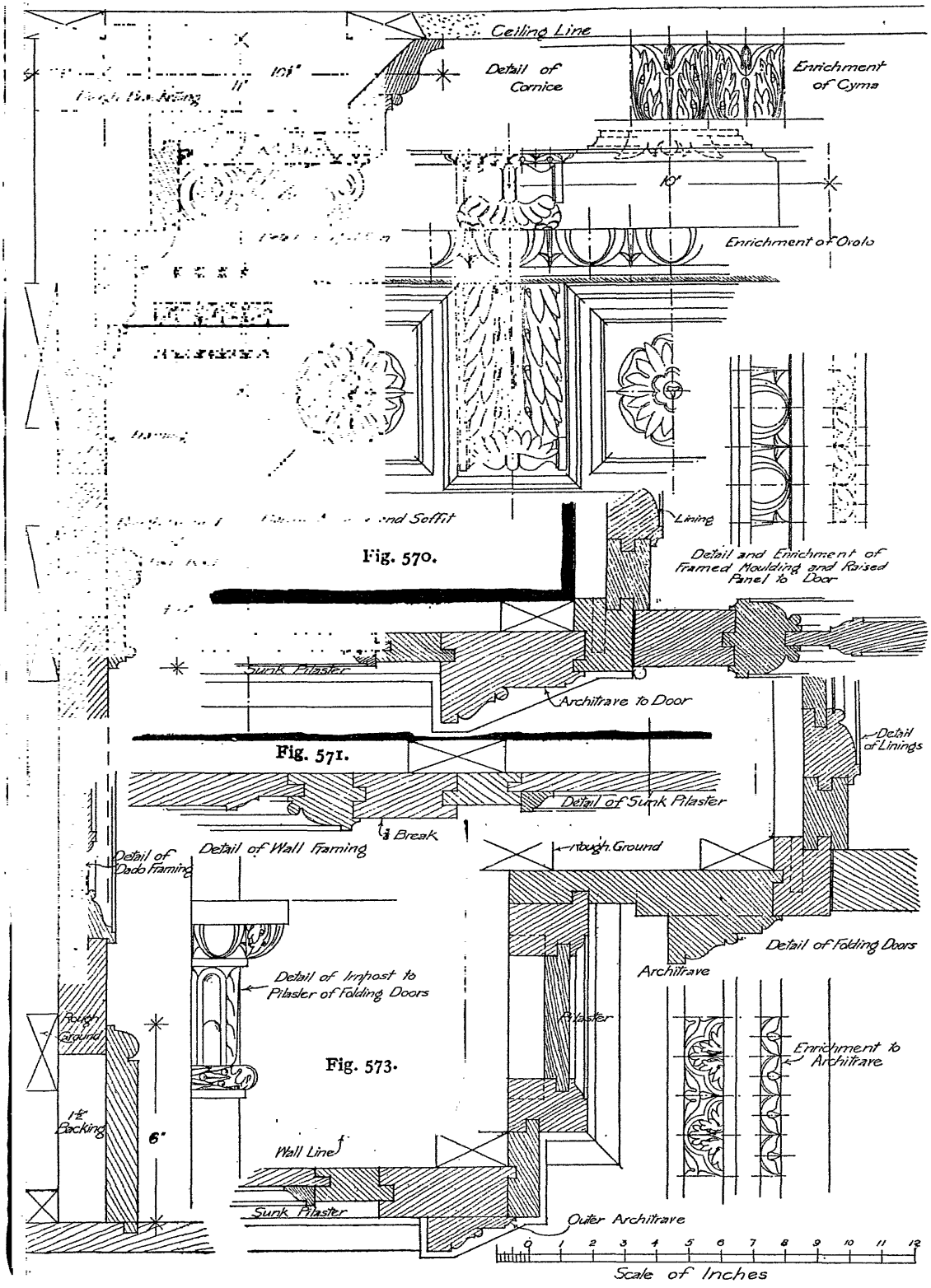


Fig. 570.

Fig. 571.

Fig. 572.

Fig. 573.

Figs. 569—573.

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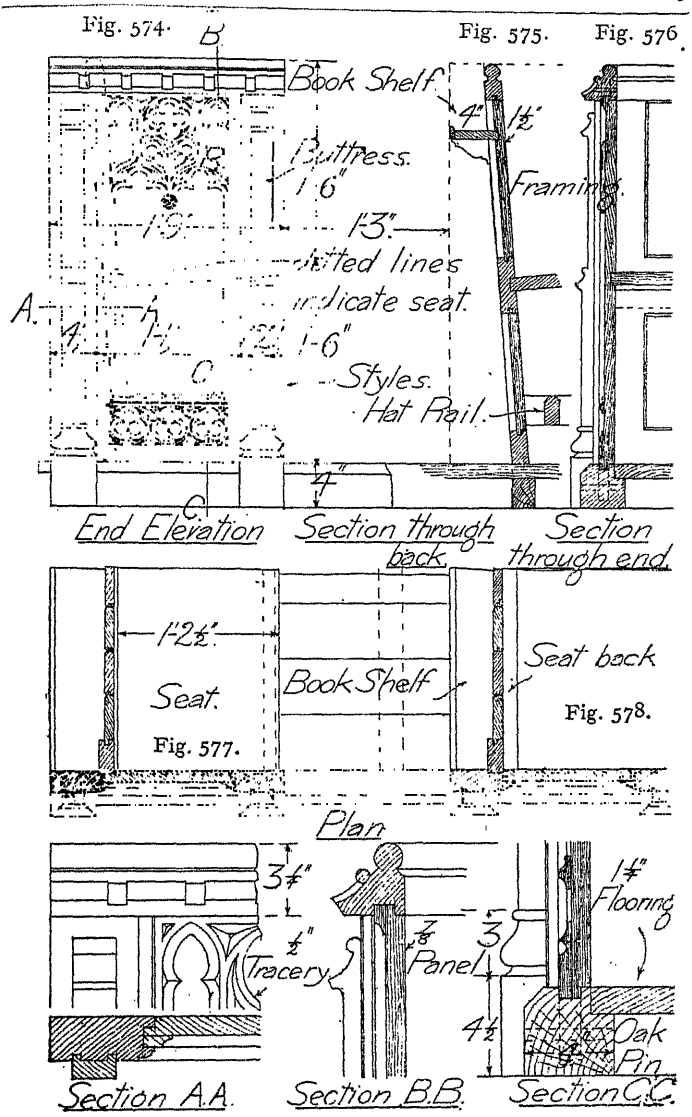


Fig. 577. Fig. 578.

Fig. 579. Fig. 580. Fig. 581.

Figs. 574—581

should slope about 1 inch in 1 foot for greater comfort. The seat should consist of one wide board, shaped on top as shown. The seat is housed into the back, and has a rail under front edge to support it, and if the seat is longer than 8 feet it should have an upright under the centre. The seat and back are housed into the pew ends. Hat rails should be provided beneath the seat, as shown in Figs. 574 and 575.

Picture Rails.—These are small moulded rails fixed about 1 ft. or 1 ft. 6 in. below the cornice, having a groove on their upper surface in which brass hooks, from which pictures may be hung, are fitted. Fig. 587 shows one fixed to the plaster wall.

Frieze.—It is not as a rule necessary to frame wood friezes. In panelled work these are fitted as panels, being grooved along their bottom edge to the architrave band of mouldings, and at their upper edge to the bottom member of the wood cornice; they have upright wood backings fixed behind them about 3 feet apart. Friezes exceeding 9 inches in width should be keyed to prevent casting. The friezes are often highly carved and painted. Where this is done, allowance must be made for the part hidden by the projection of the picture-mould or architrave, and all rails of frames and margins, where placed at a height above the eye, must be made slightly wider than corresponding rails at the height of the eye, or the adjacent vertical members, to compensate for the apparent loss in width, owing to the angle at which they are seen.

Cornices.—Wood cornices should always be built up from small sections tongued together and fixed at the pitch of the cornice, as shown in Fig. 586. Where the cornice is fixed about circular plans they should be prepared from rectangular sections as shown in Fig. 586. The heading joints should not be made across the whole section; but the pieces should be bonded together, each joint being butted, tongued or dowelled, and glued. The cornice may be made any length without loss of strength. Secondly, the mitres may be made more easily and better in small

pieces than in wide pieces. When all the pieces have been fitted, they are glued and blocked together, raised to their place in long lengths, and fixed to grounds, as shown in Fig. 570. In interiors it is a very customary practice to extend the projection of the classic cornices, as they are as a rule seen chiefly from positions more directly underneath.

Ceilings.—Ceilings are frequently covered partly or wholly with wood. (1) In the late periods of the Gothic, these consisted usually of matched planking, panelled with moulded ribs. The matched boards were either placed parallel to the walls or were arranged diagonally. The ribs were planted on the surface, and in the latter case covered the joints of the panels. For the fixing, grounds were provided along all joints or counter boarding was provided (see Fig. 582). (2) Ceilings are sometimes covered with ordinary framing. The rails, muntins and panels usually being arranged to some pattern. This method is now largely resorted to for inexpensive ceilings or in the case of reinstatement of defective plaster surfaces. A large number of wall boards have been introduced which lend themselves admirably for this type of ceiling. The joints are covered with narrow wood bands which produce a panelled surface, grounds are fixed to the underside of the joists, or to plugs driven into the concrete soffit, along the lines of the joints and in other positions for intermediate fixings where the panels are large. Then the panelled surface may be formed by planting thin narrow battens over the joints, or the battens, which may be moulded, are framed in sections to the required pattern, and are raised and fixed over the joints (see Fig. 583).

Coffered Ceilings.—These are usually employed in large ceilings in the Italian styles of decoration They are formed, as shown in Fig. 584, by branching the upper members of the cornice from the corona to the drip in various directions across the ceiling space. The ribs are built up, as shown in Fig. 586, having a number of solid wood backings to which the mouldings at the sides are fixed, leaving the soffit as a panel, free. The ribs are secured

Fig. 582.

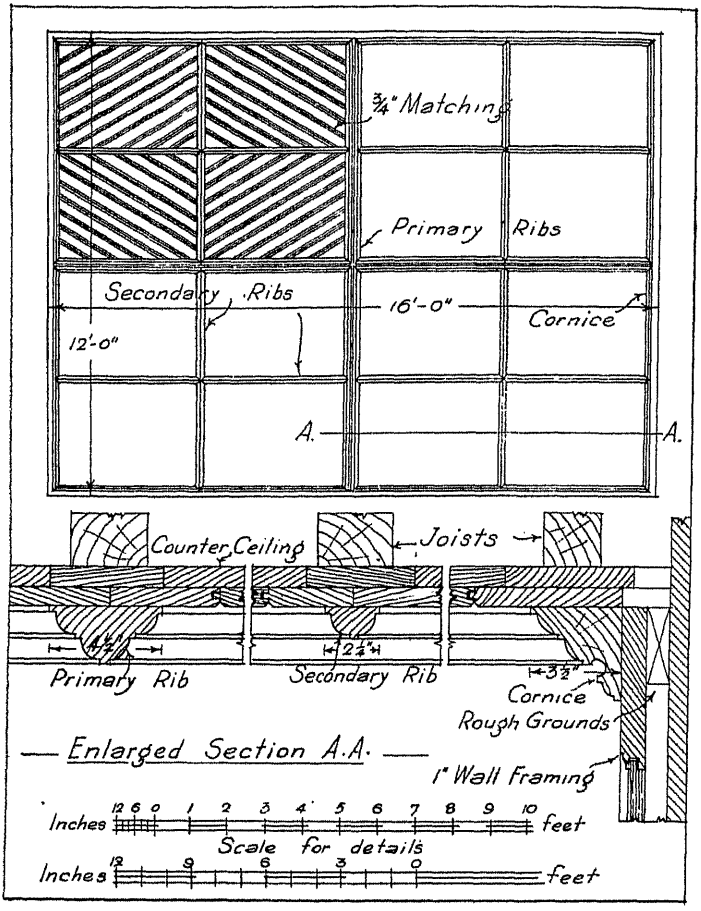


Fig. 583.

Figs. 582—583.

to rough grounds, fixed to a counter ceiling, formed of matched or plain boarding nailed to the underside of joists. The panels rest on the top members of the mouldings, and have rough grounds running at right angles to the direction

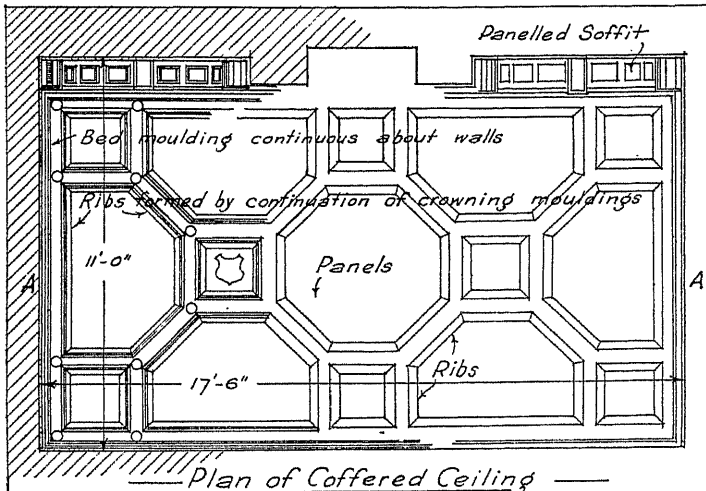


Fig. 584.

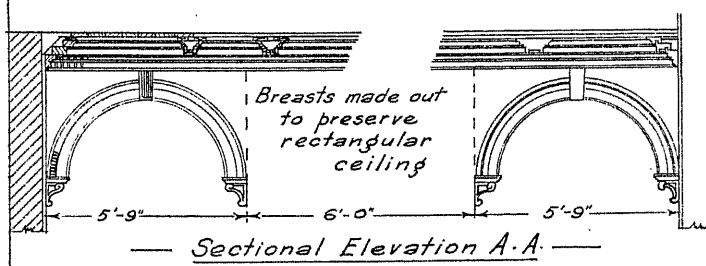


Fig. 585.

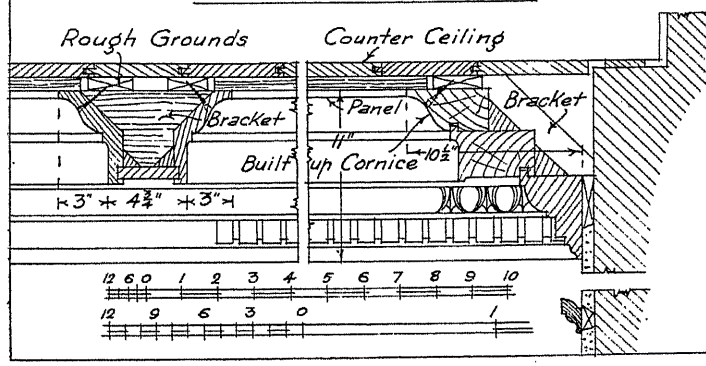


Fig. 586.

Fig. 587.

Figs. 584—587.

of their grain, to which they are fixed along their centre line in order to allow them to shrink and expand freely. These panels are often highly carved, stencilled, or painted with various designs. The narrow panels forming the soffit on the level of the drip are also ornamented by paintings, and usually have the joint at the intersections covered by a boss or patera, as shown in Fig. 584.

In order to prevent projections such as chimney breasts from interrupting the design of the ceiling, the soffit at these parts is often lowered, and built or cradled out to form an arched head, as shown on Fig. 585, and thus leaving the ceiling rectangular in plan.

Secret Fixings.—It is usually undesirable to show any nail holes upon the surface of polished hard wood, architraves, skirtings, or finishings. These finishings may be fixed (1) by means of screws which project at back of architrave or finishing, and fitting in slots in the backing, or (2) dovetailed pieces are glued to the backs of the finishings, the backings being correspondingly grooved to receive them, care being taken always to have the grain of the dovetailed projection and the grain of the member to be fixed in the same direction, as if not, under varying conditions of dryness of the atmosphere, the piece fixed is liable to become loose or split.

BRITISH STANDARD SPECIFICATION FOR DOORS

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NOTE.—*The Institution desires to call attention to the fact that this Specification is intended to include the technical provisions necessary for the supply of the material herein referred to but does not purport to include all the necessary provisions of a Contract.*

(MORTICED, DOWELLED, AND LEDGED AND BRACED) FOR INTERNAL AND EXTERNAL PURPOSES

Except where otherwise specified the figures in British measures are to be regarded as the Standard. Approximate metric equivalents are given for the convenience of users in countries in which the metric system has been generally adopted.

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1. *Scope.*—This Specification provides for the dimensions and construction of British Standard Morticed, Dowelled, and Ledged and Braced Doors for internal and external purposes, including garage doors.

2. *Dimensions.*—British Standard Doors shall conform to the dimensions given in Tables I., II. and III.

TABLE I.
Internal Doors.

1	2		3		4					
British Standard Door Numbers.	Nominal Thickness.		Thickness Min.		Width.		Height.		Width.	Height.
	in.	mm.	in.	mm.	ft.	in.	ft.	in.	Metres.	
1	1¼	31·8	1 1/16	27	2	0	6	0	·61	1·83
2	1½	38·1	1 5/16	33·3	2	0	6	0	·61	1·83
3					2	0	6	6	·61	1·98
4					2	4	6	6	·71	1·98
5					2	6	6	6	·76	1·98
6					2	8	6	6	·81	1·98
7	1¾	44·5	1 9/16	39·7	2	6	6	6	·76	1·98
8					2	8	6	6	·81	1·98
9					2	10	6	6	·86	1·98
10	2	50·8	1 13/16	46	2	10	6	6	·86	1·98

TABLE II.

External Doors.

1	2		3		4					
British Standard Door Numbers.	Nominal Thickness.		Thickness Min.		Width.		Height.		Width.	Height.
	in.	mm.	in.	mm.	ft.	in.	ft.	in.	Metres.	
11					2	6	6	6	·76	1·98
12					2	8	6	6	·81	1·98
13					2	10	6	6	·86	1·98
14	1¾	44·5	1 9/16	39·7	3	0	6	6	·91	1·98
15					2	8	6	8	·81	2·03
16					2	10	6	10	·86	2·08
17					3	0	7	0	·91	2·13
18					2	8	6	6	·81	1·98
19					2	10	6	6	·86	1·98
20					3	0	6	6	·91	1·98
21	2	50·8	1 13/16	46	2	8	6	8	·81	2·03
22					2	10	6	10	·86	2·08
23					3	0	7	0	·91	2·13

TABLE III.

Garage Doors (in Pairs).

1	2		3		4					
British Standard Door Numbers.	Nominal Thickness		Thickness Min.		Width.		Height.		Width.	Height
	in.	mm.	in.	mm.	ft.	in.	ft	in.	Metres.	
24					7	0	7	0	2·13	2·13
25					7	6	7	6	2·29	2·29
26	2	50·8	1 13/16	46	7	9	7	9	2·36	2·36
27					8	0	8	0	2·44	2·44

3. *Allowance for Milling and Dressing.*—The allowances for milling and dressing given in Table IV. shall be permitted on the nominal sizes of the door members.

4. *Timber.*—(*a*) *Quality.*—The timber from which the doors are manufactured shall be sound, bright and square

TABLE IV.

Preparation.	Width.		Thickness.	
	in.	mm.	in.	mm.
Dressed or Surfaced 4 Sides, including edges shot, ploughed and rebated	¼	6·4	3/16	4·7

edged. If natural finish or finished for staining, clear polishing or varnishing is specified the members of the door shall be uniform or symmetrical in colour.

(*b*). *Knots.*—The timber shall be free from shakes, loose or dead knots, but admitting sound knots up to 1 square inch superficial area, provided the major axis measured on the face of such knots does not exceed 2·25 inches (57 mm.). The product of the maximum and minimum dimensions measured on the surface of any sound knot shall not exceed 1·28 when measured in inches (826 when measured in millimetres).

(*c*) *Moisture Content.*—The timber shall be seasoned to the appropriate moisture content given in Table V.

Should the purchaser or his representative so specify with the enquiry and order, a test for moisture content shall be carried out in the manner described in Appendix I.

(*d*) *Bright Sapwood.*—Bright sapwood shall be permissible in soft woods only and if present shall not exceed the limit specified in Table V. when measured as a percentage of the superficial area at any cross section of the timber.

TABLE V.

Moisture Content and Bright Sapwood Limits.

Type of Door.	Moisture Content for Soft and Hardwoods. Max. %	Permissible Bright Sapwood Content Softwoods. Max. %
Internal	12	12½
External (including Garage Doors) ...	18	12½

5. *Panels.*—(*a*) *Type.*—Panels for external doors shall be solid, and shall comply with the requirements laid down in (*b*) below.

Panels for internal doors may be either solid or of plywood, as may be specified, and shall comply with the requirements set out in (*b*) or (*c*) respectively.

(*b*) *Solid Panels.*—The thickness of solid panels shall be not less than one-third the finished thickness of the door. The panels shall be truly square and shall be framed in grooves to a depth not less than $\frac{3}{8}$ inch (9·5 mm.). Where bead butt or bead flush panels are specified the thickness shall be increased as required.

(*c*) *Plywood Panels.** (i.) *Quality.*—Panels shall be of sound wood † and shall be of a quality and finish corresponding to that of the adjoining members of the door. When natural finish or finished for staining, clear polishing or varnishing is specified they shall be uniform or symmetrical in colour with each other and with the adjoining surfaces of the door, and the grain of the outer plies (except in lay panels) shall be generally parallel to the door stiles. Panels shall be truly square and shall be not less than 6 mm. (0·24 inch) in thickness and framed in grooves to a depth not less than $\frac{3}{8}$ inch (9·5 mm.).

(ii.) *Adhesion of Plies.*—The adhesion of the plies shall be tested by forcibly separating the layers. The plies shall offer appreciable resistance to separation and the fractured surfaces shall show some adherent fibres distributed more or less uniformly.

(iii.) *Resistance to Water.*—The resistance to water of the plywood shall be such that, when tested in the manner described in Appendix II., the samples shall still comply with the requirements specified in para. (ii.) above and shall show no appreciable signs of separation at the edges of the plies, or formation of blisters.

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* A B.S. Specification for Commercial Plywood is in course of preparation.

† For the purposes of this Specification the term " sound wood " denotes wood free from fungal or insect attack.

6. *Framing.*—(*a*) *Methods.*—Doors shall be framed by joining the members where they intersect by means of mortice and tenons or of dowels, as may be specified, in such a way as to secure rigidity and soundness throughout the frame.

(*b*) *Tenons and Haunchings.*—The thickness of each tenon shall be approximately one-third the thickness of the door and the width of each tenon shall not exceed five times its own thickness. Haunchings shall be sunk to a depth not less than $\frac{3}{8}$ inch (9·5 mm.) and in order to give a good fit they shall be clear of the bottom of the groove by a distance not greater than $\frac{1}{16}$ inch (1·6 mm.) before jointing.

(*c*) *Dowels.*—Dowels shall be straight-grained and keyed for gluing.

The minimum size of dowels for all thicknesses of doors shall be $\frac{5}{8}$ inch (16 mm.) dia. × $4\frac{7}{8}$ inch (124 mm.) in length. They shall be equally divided in the adjacent members and shall be placed at distances not greater than $2\frac{1}{4}$ inches (57 mm.) centre to centre.

(*d*) *Scribes and Tongues.*—When doors are dowelled a continuous machine scribe and/or tongue shall be employed at the shoulders of all members.

The tongue left between the machine scribes shall be sunk to a depth of not less than $\frac{3}{8}$ inch (9·5 mm.) and in order to give a good fit it shall be clear of the bottom of the groove by a distance not greater than $\frac{1}{16}$ inch (1·6 mm.) before jointing.

(*e*) *Gluing.*—The contact surfaces of dowels, mortices, tenons, and shoulders shall be treated, before putting together, with one of the following adhesives, which shall conform to the appropriate British Standard Specification:—

Gelatine Glue	B.S.S. No. V. 11.
Liquid or Jelly Glues... ...	B.S.S. No. V. 10.
Casein Cement...	B.S.S. No. V. 2.

7. *Moulds and Beads.*—Moulds and beads unless planted shall be scribed at the joints where this is practicable. In the case of moulds planted and nailed the [illegible] kept clear of the panels.

8. *Finish.*—British Standard Doors shall conform to the following requirements on delivery :—

They shall be truly square when tested by measuring the diagonals taken from the inner intersection of the rails with the stiles and shall be flat when tested by means of twisting sticks. They shall conform truly to a straight edge wherever applied and the finished surfaces of the door shall be flat, smooth, free from machine marks and if natural finish or finished for staining, clear polishing or varnishing is specified they shall be free from cross sand-paper marks.

9. *Framed, Ledged and Braced Doors.*—(*a*) *Description.*—For the purposes of this Specification the term " Framed, Ledged and Braced " applies to doors framed in skeleton, panelled after framing together and then braced. They shall conform to the requirements of this Specification, where applicable, and to the requirements laid down in (*b*) and (*c*) below.

(*b*) *Bracing.*—When doors are braced, the braces shall be so fixed that their lower ends are adjacent to the hanging stile of the door.

(*c*) *Coverings.*—Coverings shall be made up of boards tongued and grooved together and securely fastened to the frame and braces. If required for external work they shall be painted together at the joints and adjoining surfaces of the frame and contact surfaces of the ledges and braces with a paint having a red or white lead base conforming to the requirements of B.S.S. Nos. 217 or 239 respectively.

The minimum finished thickness of coverings shall be not less than $\frac{5}{8}$ inch (16 mm.).

10. *Ledged and Braced Doors (Batten Doors).*—(*a*) *Description.*—For the purposes of this Specification

the term " Ledged and Braced " applies to doors composed of a single covering made up of boards tongued and grooved and securely fastened to the ledges (battens) and braced.

They shall conform to the requirements of this Specification, where applicable, and to the requirements laid down in (*b*), (*c*) and (*d*) below.

(*b*) *Ledges* (*Battens*).—Not less than three ledges shall be employed. These ledges shall be not less than $4\frac{1}{2}$" × $1\frac{1}{4}$" (114 mm. × 31·8 mm.) nominal in cross section, and shall be weathered at both edges to an angle of 60 degrees to the face of the door.

(*c*) *Bracing*.—When doors are braced the braces shall traverse diagonally the back of each panel formed by the ledges, and shall be so fixed that their lower ends are adjacent to the hanging edge of the door. These braces shall be not less than $4\frac{1}{2}$" × $1\frac{1}{4}$" (114 mm. × 31·8 mm.) nominal in cross section and shall be weathered at both edges to an angle of 60 degrees to the face of the door. If the braces are required to be housed into the ledges, this shall be specified in the enquiry and order.

(*d*) *Coverings*.—Coverings shall be made up of boards tongued and grooved together and securely fastened to the ledges and braces. If required for external work they shall be painted together at the joints and contact surfaces of the ledges and braces with a paint having a red or white lead base conforming to the requirements of B.S.S. Nos. 217 or 239 respectively.

The minimum finished thickness of coverings shall be not less than $\frac{5}{8}$ inch (16 mm.).

APPENDIX I

Method for the Determination of Moisture Content

(*a*) *Selection of Samples*.—For the purpose of carrying out the test for moisture content (see Clause 4(*c*)) the purchaser or his representative may select, during the

course of manufacture, suitable samples of the timber used in the construction of the doors.

If the test for moisture content is to be carried out immediately after the cutting of the sample, these samples may take the form of the test pieces specified in (*b*) Method of Test; but if the first weighing cannot be carried out immediately after the cutting of the sample, the samples shall consist of pieces cut from the stock not less than 18 inches (457 mm.) in length from which the final test pieces shall be cut when the test is made.

(*b*) *Method of Test.*—The test pieces of the timber shall be taken at a point not less than 9 inches (229 mm.) from the end of the sample, and shall be cut to include the full cross section of the sample and shall be $\frac{3}{4}$ inch (19 mm.) long in the direction of the grain. They shall be weighed (W_1) immediately after cutting, and then dried in an oven at a temperature of 212° to 221° Fahr. (100° to 105° Cent.) until the weight is constant (W_0) and again weighed immediately after removal from the drying oven.

The percentage moisture content (X) shall then be determined from the formula :—

$$X = \frac{W_1 - W_0}{W_0} \times 100.$$

APPENDIX II

Method for the Determination of Resistance of Plywood to Water

(*a*) *Test Pieces.*—Two test pieces shall be cut from one board out of every twenty-five forming a consignment. Each test piece shall be approximately 9 inches (229 mm.) long by 4 inches (102 mm.) wide cut parallel to the length of the board.

(*b*) *Method of Test.*—The test pieces shall be immersed in water at a temperature of 212° Fahr. (100° Cent.) for a period of 3 hours. At the end of this period one of the test pieces shall be removed from the water and plunged immediately into cold water. After cooling to a temperature of 60° Fahr. (15·6° Cent.) in the water, the test

Fig. 588. Fig. 589.

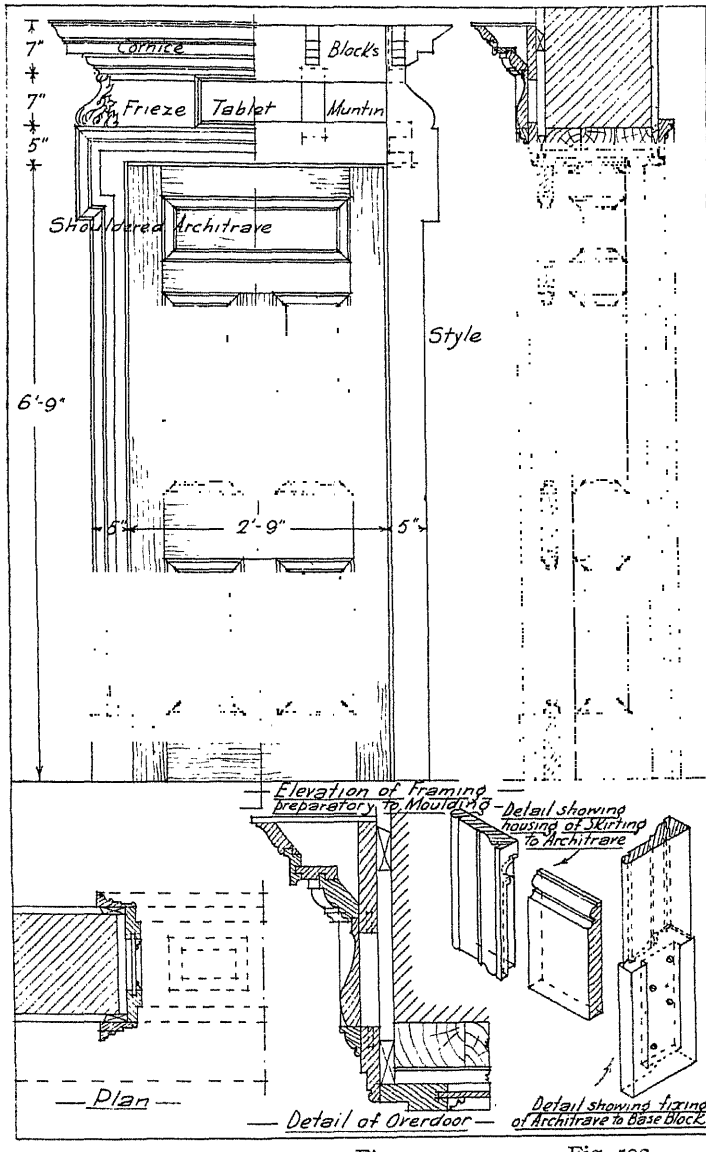


Fig. 590. Fig. 591. Fig. 592.

Figs. 588—592.

piece shall be removed from the water and whilst still water-soaked shall be tested for compliance with the requirements specified in Clause 5 (*c*), para. (iii.).

The other test piece, on removal from the boiling water, shall be allowed to dry for 72 hours at a temperature of 60° Fahr. (15·6° Cent.) and shall then be examined for signs of separation at the edges of the plies, or formation of blisters (see Clause 5 (*c*), para. (iii.)).

Doors.—Doors and door linings with the methods of framing and hanging have been treated in the *Elementary Course.* Some special cases and treatments are given in this work. Doors, owing to their continual movement, require special care in their framing and in the selection and conditioning of their material or they will invariably warp, twist and shrink, in which case a good fit is impossible. The British Standard Specification gives the salient points that should be looked to in their construction.

Figs. 588 to 592 give an example of a five-panelled door, the top lay panel usually being known as the frieze panel. These figures show a shouldered architrave and overdoor, one half the elevation showing the groundwork for the mouldings and the building up of the entablature, also details of the construction of the base block to the architrave. Figs. 593 to 596 give a second example of an internal doorway, with a pair of folding doors. Where it is necessary to have openings over 3 feet in width, folding doors should be supplied, first to reduce the stress on the hinges and second to reduce the plan space required for the door swing. This example shows the architrave surround emphasized by side pilasters and an overdoor, the cornice being supported by consoles. One half the elevation shows the groundwork framing upon which the moulded work is built.

Figs. 597 to 602 show an example of internal folding doors which, if constructed in hard wood, would be very heavy; in such cases solid frames are required to afford the necessary screwhold for fixing the hinges. Fig. 602 gives details of the construction of the linings, which are framed up with the posts; the latter are either tenoned to

Fig. 593. Fig. 594.

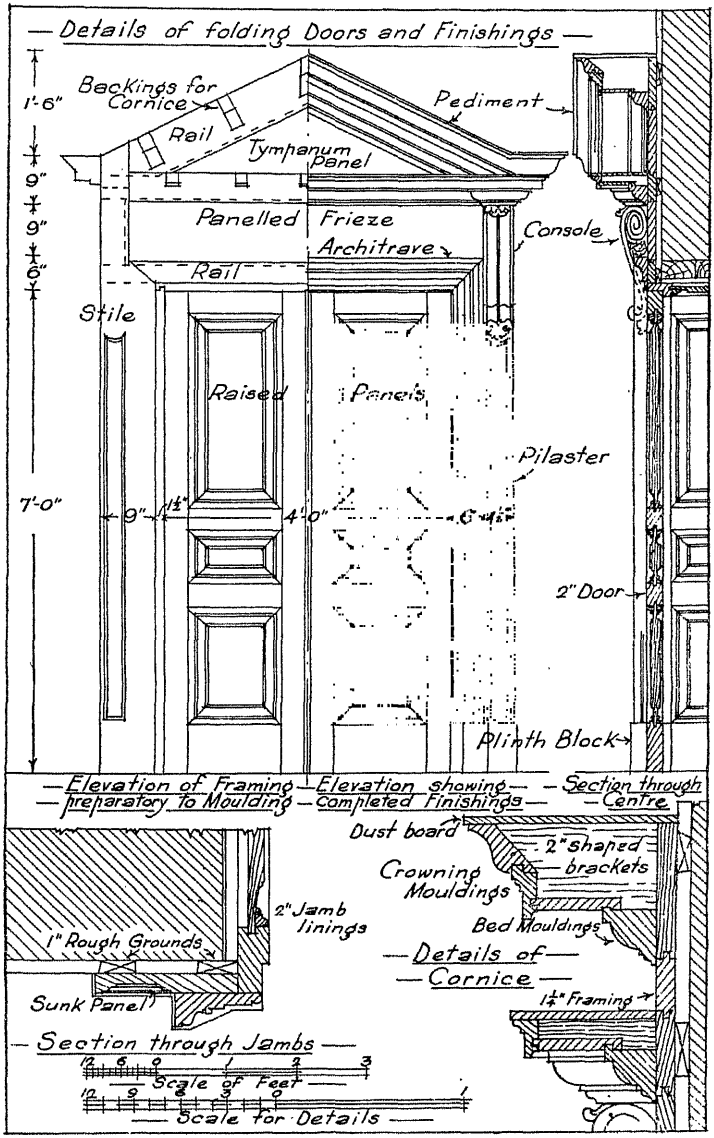
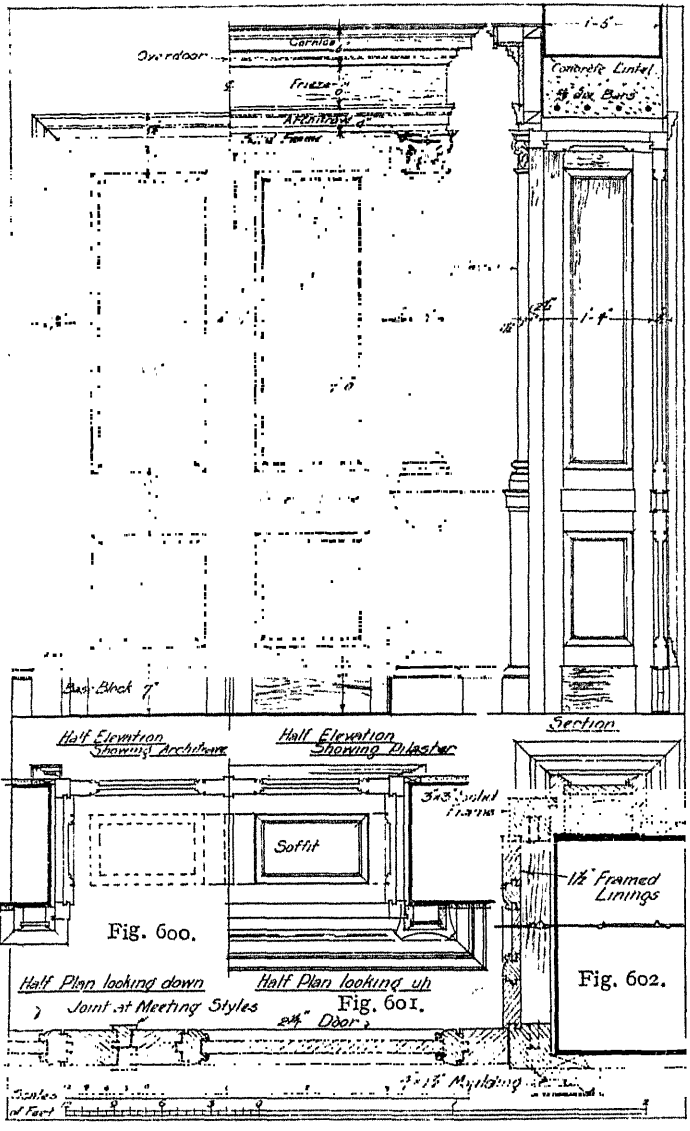


Fig. 595. Fig. 596.

Figs. 593—596.

Fig. 597. Fig. 598. Fig. 599.



Figs. 597—602.

the floor, or if the floor is of concrete or stone it is fixed at that point with metal dowels. The opening is shown finished with pilasters and an overdoor on one side and plain architraves on the reverse.

Figs. 603 to 609 show an external door with details of a circular canopy supported on brick pilasters and stone consoles. The door here is shown as a double margin door, an expedient employed where a single door would appear out of scale, but where internal requirements only admit of one door. Such doors are prepared in two leaves. The meeting styles are butt jointed, the edges being rebated and sunk beads inserted to cover the joint. The joint, which is glued, is further secured by pairs of hard wood folding wedges, glued and passed through mortices made in the meeting styles to receive them. See Figs. 603 to 604. After the door is fitted in the frame an iron bar is sunk in the top edge of the door, to further secure it at this part.

Figs. 610 to 613 show a general view of a pair of external doors with lobby and swing doors. The front doors are shown to fold back into a recess when the premises are open, and appear as framed linings at the side of the lobby; alternate methods are shown of forming the hanging style. In Fig. 618 the door when open would be flush with the frame, the joint being broken with a bead of the same dimension as the knuckle of the hinges employed. Fig. 616 shows a simpler arrangement: in this case the door, when open, is set back from the face of the frame. Fig. 615 shows a large roll moulding on the hanging style which fits into a corresponding hollow on the frame. The door is flush with the frame when open, the joint being masked by the roll moulding. In this case the door rotates on centres at bottom and top. The inner swing doors are fitted into a solid frame. The hanging styles of the doors are worked to a segment of a circle which fits into a corresponding hollow on the frame. The centre of rotation should be at least five eighths of the thickness of the door from the face of the frame, to allow the doors to open at right angles to the frame, see Fig. 614. The meeting styles must also be rounded—the curve for these is struck from the centre of

Fig. 603. Fig. 604. Fig. 608. Fig. 605.

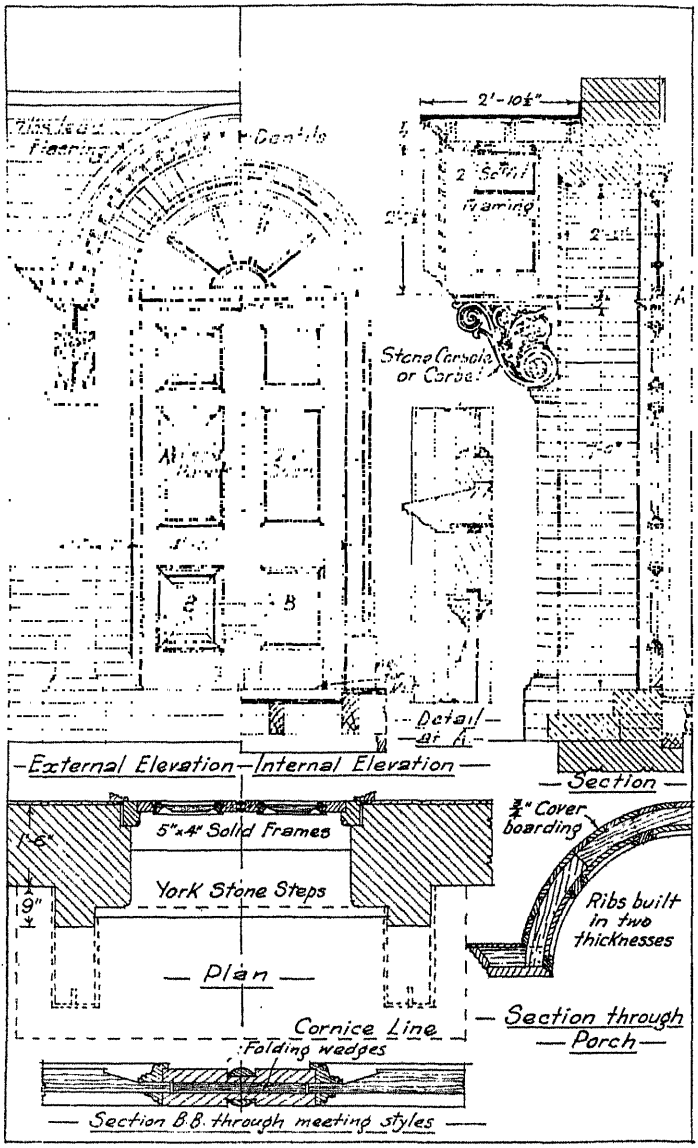


Fig. 606.

Fig. 609. Fig. 607.

Figs. 603—609.

Fig. 610. Fig. 611. Fig. 612.

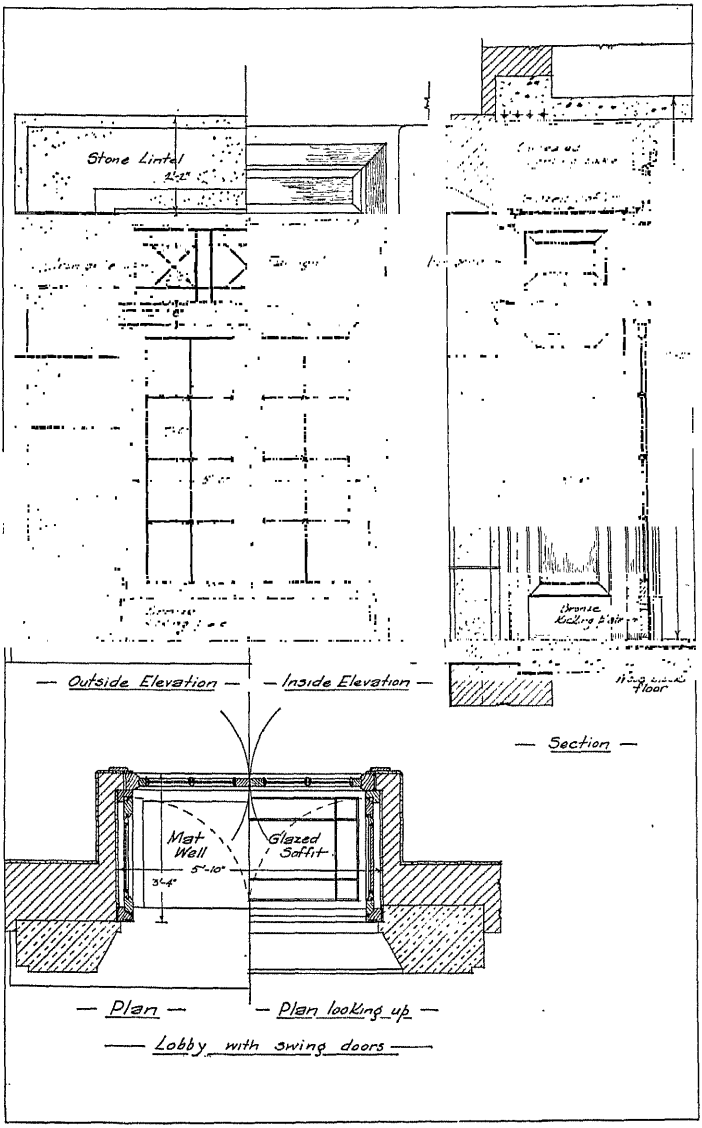


Fig. 613.

B.C. Figs. 610—613. E E

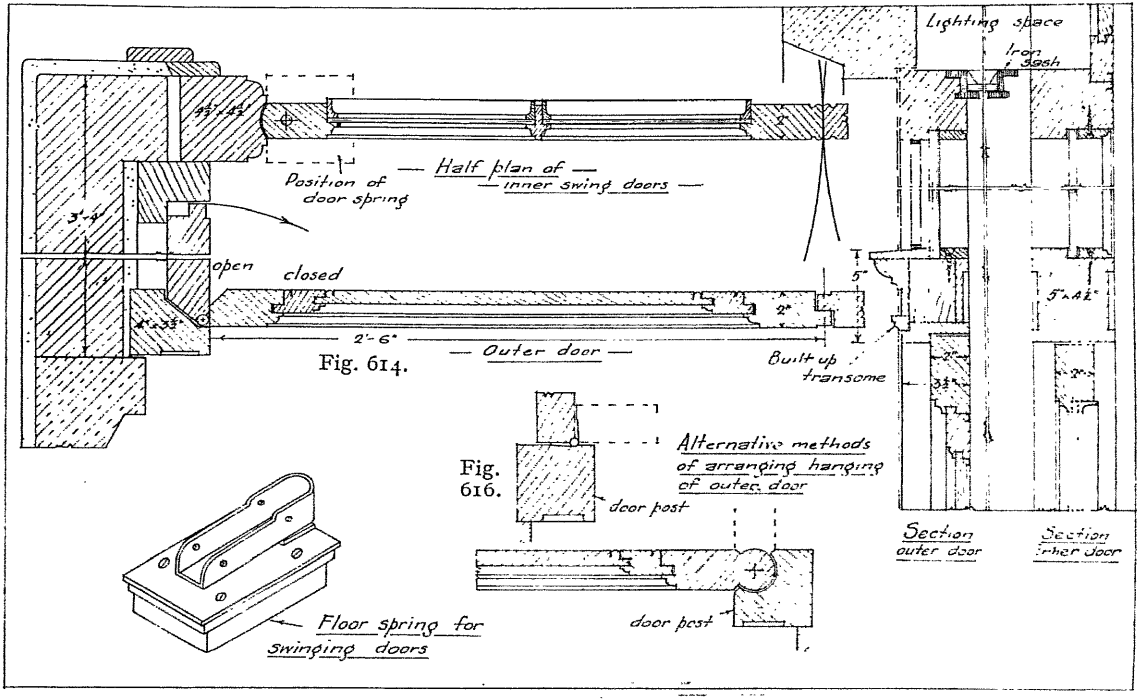


Fig. 614.

Fig. 616.

Fig. 617.

Fig. 615.

Fig. 618.

Fig. 619.

Figs. 614—619.

rotation. These doors swing on centres; the lower one works in co-operation with a spring contained in a box that is let into the floor; on the pivot is mounted a heel socket that encloses the lower angle of the door; the upper centre is made to rise and fall to enable the door to be mounted. When the spring is at rest, the door is closed. The upper parts of the frames show fanlights and the soffit of the lobby is formed with a steel light usually glazed with ground or rippled glass; above this the artificial lighting is fixed. The lobby can thus be brilliantly lit without glare, see Figs. 612, 618 and 619.

Revolving doors.—This type of door is very suitable for entrance halls to such buildings as clubs, hotels, restaurants, etc. Its special claim is that it is draught excluding. At no time in the revolution of the four leaves is there any direct communication between the outside and the inside as is the case with ordinary doors with which, when in use, if there is a big difference in temperature between the outside and the inside, there is a big rush of air through the opening and a great loss of heat. The arrangement consists of four leaves connected in various ways according to the patent at their pivot styles. These are enclosed in a circular framework. There is a clearance of about half an inch along the outer edges of the leaves for safety, but this space is closed with flexible strips of rubber fixed with a fillet screwed along the outer styles. The four leaves are made to collapse and fold against one side if a clear opening is required. The enclosing framework, if with solid panels, must have the latter flush on the inside, so that the rubber strips have a perfectly plain surface to rub against. If this enclosure is glazed the face of the glass must be flush with the face of the framework. The glass in this case is kept in its place with strips of metal, brass, bronze or aluminium of a thin gauge on the inside face. These are the essential points. The actual pivots and centres vary and are the subject of patents by the various specialist firms that make these doors. For closing the establishment, it is usual to have revolving shutters or collapsible gates to shut off communication with the street, see Figs. 621 and 622.

Fig. 620. Fig. 621.

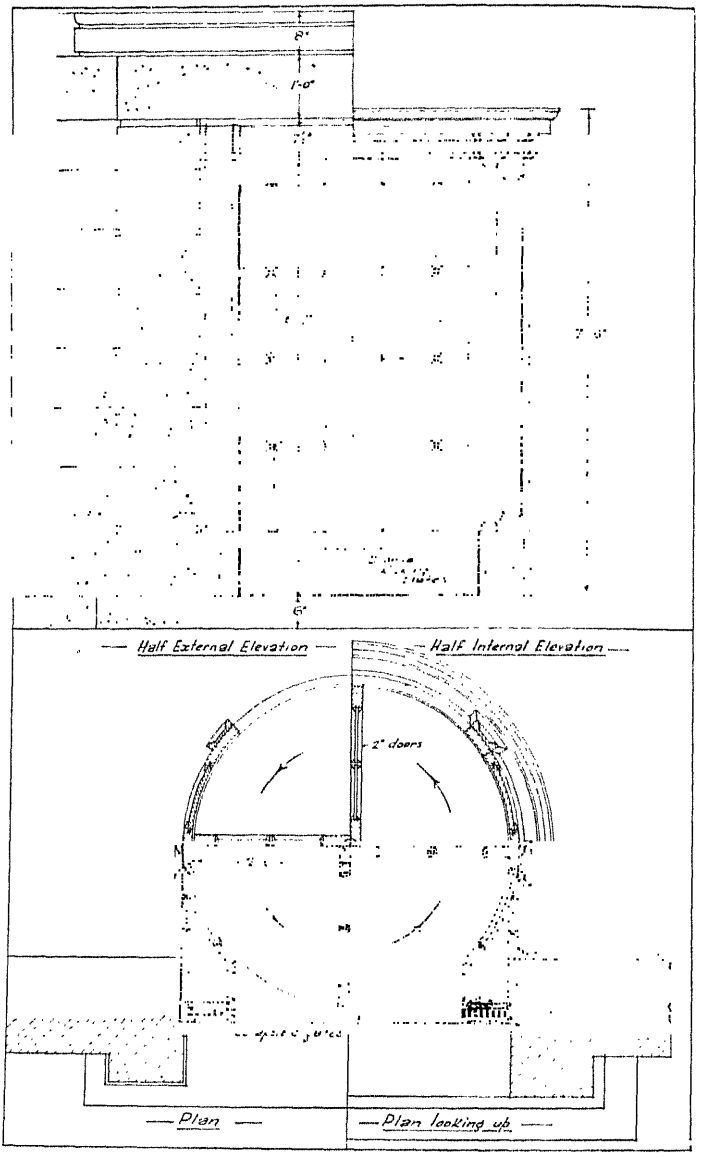


Fig. 622

Figs. 620—622.

Flush doors.—There is an objection to framed doors, especially if they are highly moulded or carved, that they offer numbers of ledges and indentations on and in which dust can collect and from which it is not easily dislodged. For many purposes such as hospital wards, and bedrooms, such dust traps are a menace and detract from the hygienic efficiency of rooms of this type. For such purposes the flush door is the solution, but for doors for any purpose the

Fig. 623. Fig. 624. Fig. 625.

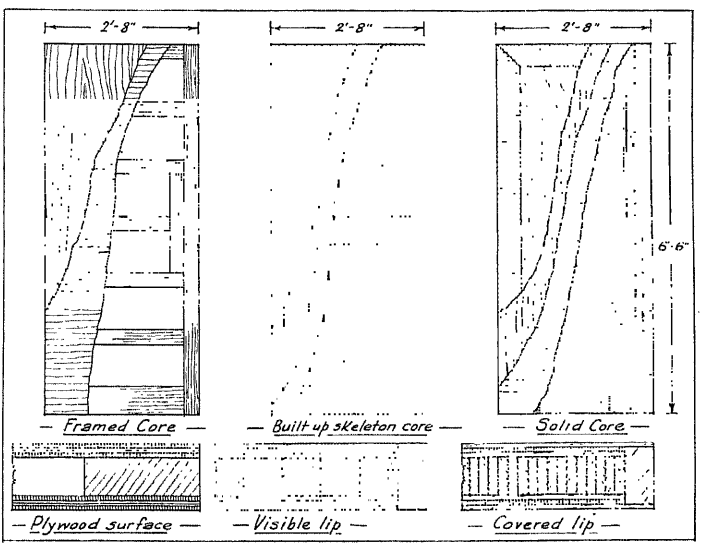


Fig. 626. Fig. 627. Fig. 628.

Figs. 623—628.

beautiful effects obtained by veneering, especially when the latter is arranged in patterns, are superior to most framed work; this is most noticeable where highly figured woods of rich colouring are employed. These doors are formed in two ways. (1) A light frame is prepared on each side of which a surface of plywood is cemented, the whole being subjected to great pressure in presses; (2) a solid core of wood laminæ is first prepared. The laminæ

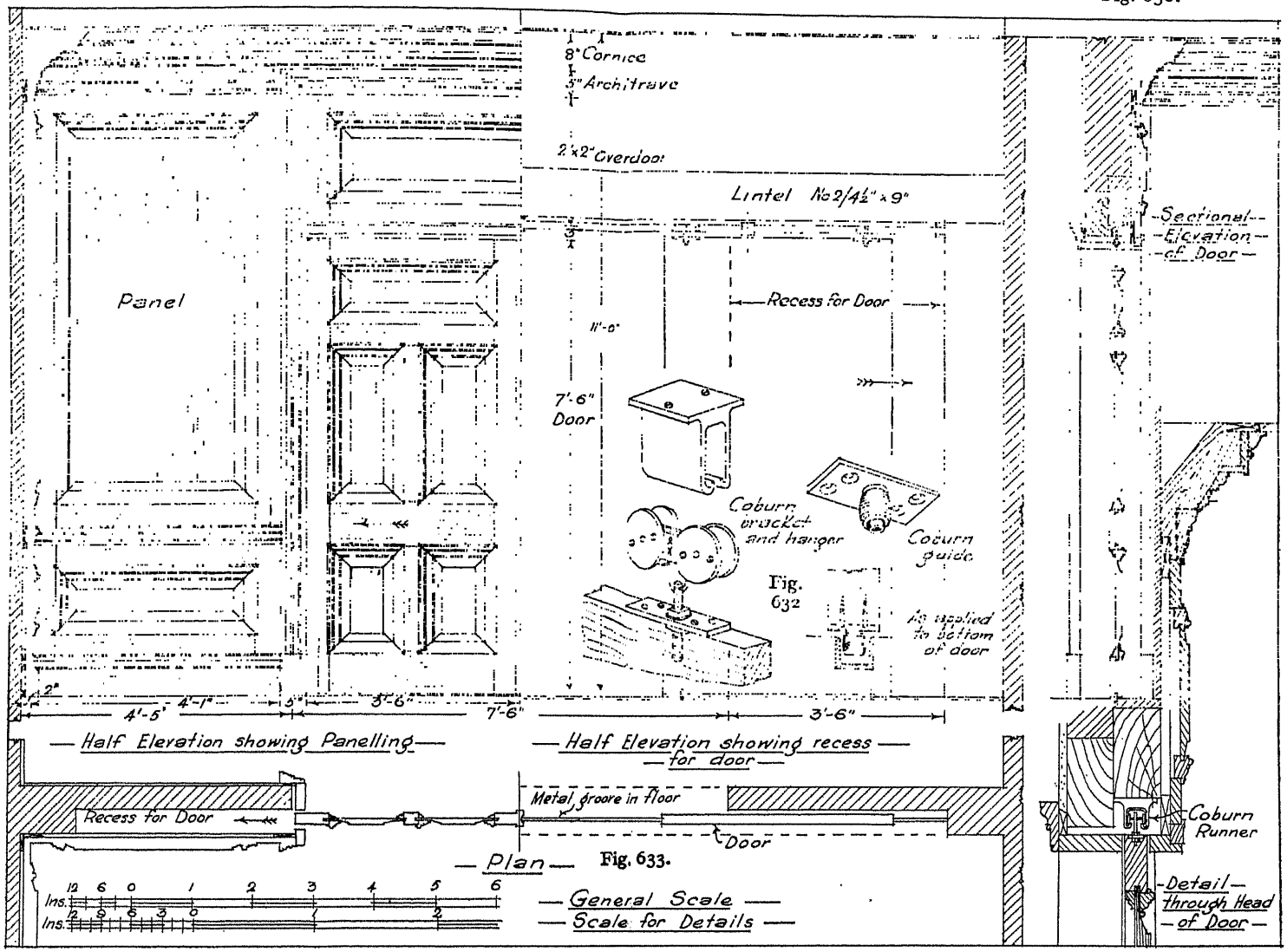
are prepared from straight-grained wood cut in thin strips. The strips are carefully dried to about a 12 per cent. moisture content; the grain of each piece is then reversed, so that any tendency of the laminæ to warp is prevented by their opposite tendencies. These strips are cemented and pressed together; after drying the surfaces are trued and veneered with at least two sheets of veneer on each side; the first layer is laid with its grain at right angles to that of the core, and the final layer is cemented with its grain in the same direction as the core. These doors when properly made are exceedingly strong and do not warp, twist or shrink; the outer edge of the door is usually lipped in order that the edges of the core and plywood or veneered surfaces shall not be seen. The hanging edge is not usually lipped but left for fitting. These doors may be obtained in the stock sizes and up to 2 inches in thickness. There is a very large range of beautifully coloured woods imported from Empire sources for polished finishes, also a large number of plainer woods for painting. Lamina boards can be obtained in thicknesses from $\frac{3}{8}$ inch to 2 inches. The thinner sheets are mostly employed for wall sheeting, and thicknesses from $\frac{3}{4}$ inch to 2 inches for doors. The sheets for lining walls can be obtained in dimensions up to $5' \times 15'$.

Figs. 623 to 628 show methods of forming room doors in this material.

Sliding Doors.—This method of hanging is employed for wide doors, where the space required for them to revolve on one edge is not available. It is chiefly employed for factory doors or gates, but often for residential buildings. The apparatus is as follows: An iron bar, sufficiently strong to support the doors without deflection, is fixed to the wall, usually by having the two ends turned at right angles and built into it. If there is to be a pair of doors, the bar is also supported in the centre by a piece of iron of similar section, fixed to it at right angles, and projecting into the wall at that part. The bar is rounded at the top edge; and on this wheels with a hollow edge revolve, the wheels being fixed at their centres with iron straps connected to the doors below. Where the appearance of the doors

Fig. 629.

Fig. 630.



Figs. 629—633.

[*Between pages* 838 *and* 839.

is a matter of consideration, the straps must be fixed to the top edge of the door; but for factory and that class of door, where strength is chiefly required, the straps, which are in pairs, are let into or fixed upon the face of the doors on each side, bolts being passed through the two straps and the door.

Internal Sliding Doors.—These are largely used for wide openings, where it is inconvenient to have hanging doors. Sliding doors are employed in order to save the considerable amount of floor space required for the door swing necessary for the latter.

Figs. 629 to 633 show the arrangements for a pair of sliding doors between two rooms, divided by a 9-inch brick partition. A recess, half a brick deep, is formed in one side of the wall sufficient to contain the doors when open. The doors are shown fitted with Coburn runners. The runners are formed of steel of the section shown on Fig. 632. Each door is suspended from two points on the top edge from rollers which roll in the runner grooves. Each roller has four wheels mounted on ball bearings, and has adjusting screws to allow the door to be centred easily. The runners are supported by special cast brackets which may be bolted to the lintel above, or formed to be bolted to the face of the lintel. In the floor is a metal groove. In the bottom of the door projecting studs are fixed to run in the groove, thus preventing any swaying of the door in a lateral direction.

The walls in the room (Fig. 629) are shown panelled; in order that the panelling may be easily removed to admit of adjustments being made to the sliding gear, it should be fixed with screws. The illustration shows the general arrangement of the parts.

Windows and Finishings.—The usual forms of sashes and frames, with the simpler types of finishings, have been treated in the *Elementary Course*. Windows, apart from their fundamental use for lighting interiors, also provide facilities for illegal entry. To hinder the latter in property containing articles of value shutters of some type are usually incorporated with the finishings. Of these there

Fig. 634. Fig. 635. Fig. 636.

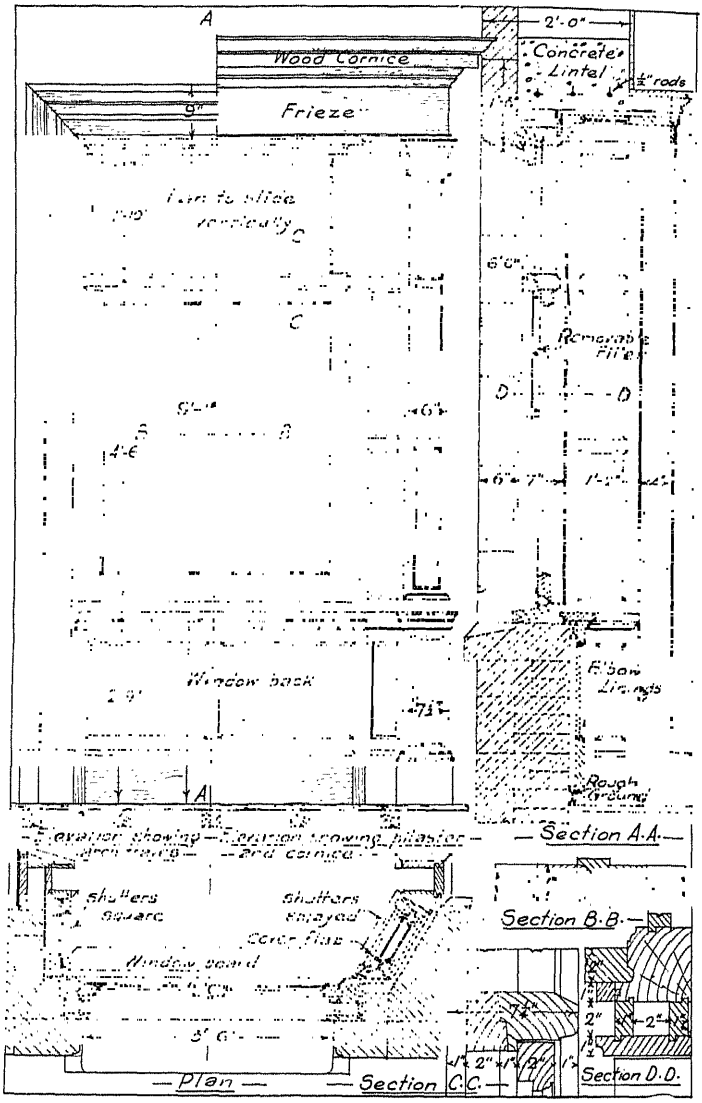


Fig. 637. Fig. 638.

Figs. 634—638.

are three general types. The boxed, the lifting, and the rolling shutter.

Figs. 634 to 638 show a type of window combining the casement and lifting sash. The frame has a solid head and sill. The side posts are made sufficiently wide to satisfy the requirements of the casement doors fitted in the lower part of the frame. The fanlight instead of being hinged at top or bottom is made to slide vertically in a casing formed on the outside of the solid posts, see Fig. 638. This provides a safe type of window for cleaning purposes, especially when in lofty positions. Boxing shutters are shown alternately splayed and at right angles with curtain boxes on the inside, one half shown with an architrave and the other side with an overwindow and pilasters.

Figs. 639 to 643 show a double hung sash finished internally with lifting shutters. In this case the principle is similar to the double hung sash, but with solid shutters instead of sashes. The cased frame is carried down to the floor. With lofty windows it may be necessary to provide three or more sashes, to enable the shutters to be accommodated between the floor and the sill level. At the sill level a flap is hinged to the window back. When closing the shutters the flap is opened, the shutters are raised, the flap is then closed and the lowest shutter is lowered till it is in contact with the flap, see Fig. 641.

Figs. 644 to 647 show a double casement, the object of which is to reduce the passage of sound. In urban districts where there is a large volume of traffic, the amount of noise passing through the windows if single, is liable to be very distracting, and considerably reduces the efficiency and value of such premises for office purposes. The addition of a second set of sashes on the interior will very considerably reduce the volume of sound passing into the building from this source. Where double sashes are employed, ventilation must be provided from other sources than open windows.

Figs. 645 and 646 show revolving shutters. Wood shutters afford no real deterrent to the enterprising burglar, unless they are abnormally thick and of hard wood. The hard steel corrugated sheets or slabs are much more difficult to perforate. Figs. 645 and 646 show the shutter

Fig. 639. Fig. 640.

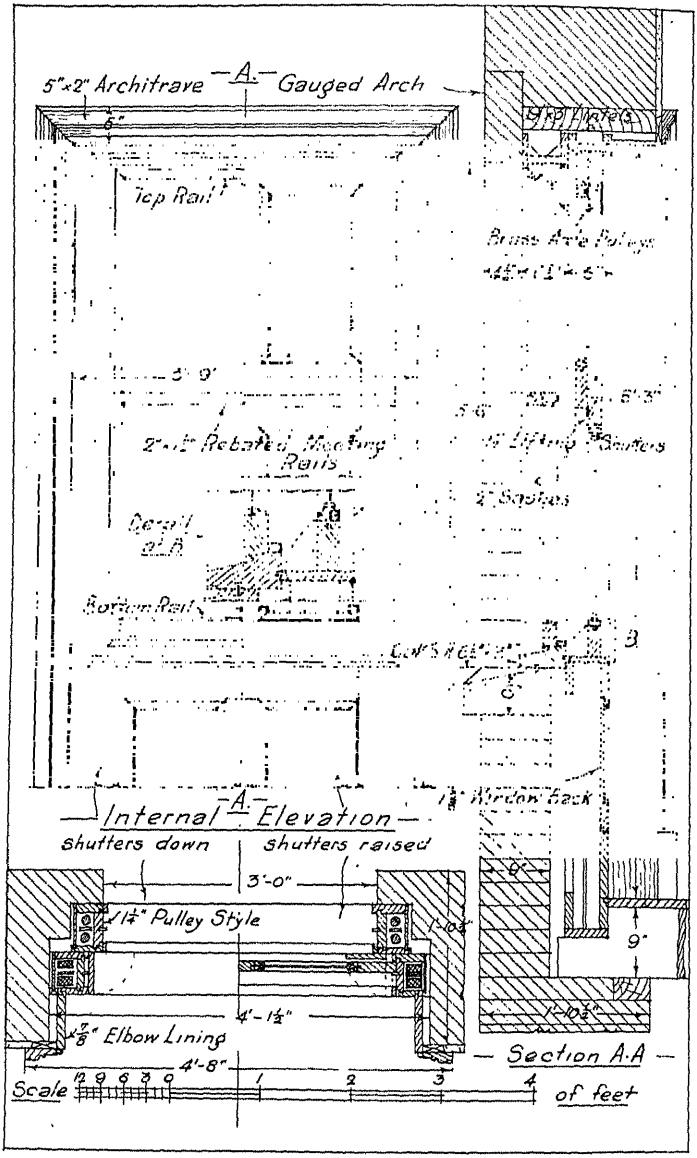


Fig. 643.

Fig. 642. Fig. 641.

Figs. 639—643.

Fig. 644. Fig. 645. Fig. 646.

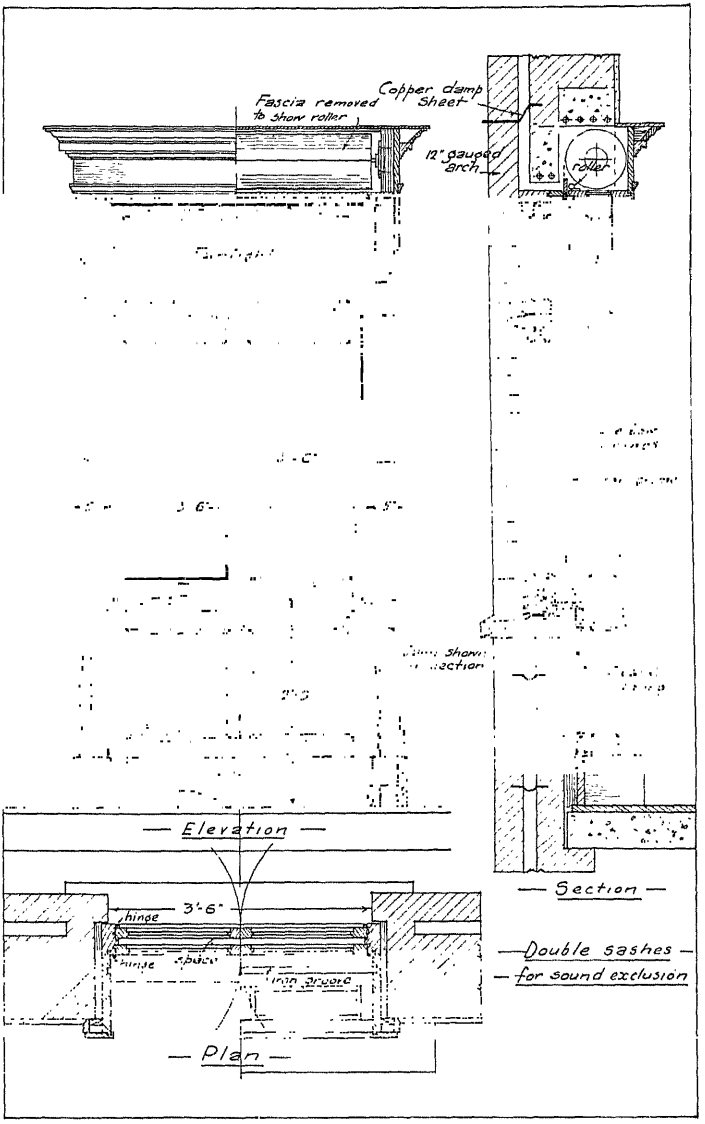


Fig. 647.

Figs. 644—647.

enclosed in a boxing arranged just below the lintel; the shutter passes through the head lining and runs down iron grooves sunk in the elbow linings, until the steel slat at the bottom edge of the shutter rests on the window board. The shutter is fixed to a steel cylinder containing spiral or helical springs, which always tend to keep the shutter in the open position. To close the shutter, use a long arm with a hook at the end, which usually fits into a loose ring provided for the purpose in the bottom slat. The shutter coil is mounted on two square cast iron sockets or on special cast brackets. Shutters of this type are extremely efficient and take up very little room, and do not interfere with blinds or curtains.

Revolving shutters can be placed in a box behind the window board. The box is made to form a seat by the window. This arrangement is not so good as the former, as the shutter has to be lifted. There is the weight of the shutter, about 5 lbs. per super foot, and the pull of the spring to be overcome. This requires some gearing to wind it up or counter weights each side, and makes a relatively complicated job compared with the pull down arrangement. Where revolving shutters are contemplated the makers should be consulted before setting out, as there are slight variations in the details.

Skylights are sashes fixed on pitched roofs, as shown in Figs. 648 to 654, primarily to light the space below. They are also often made to open for ventilating purposes, the process of fixing being as follows: The common rafters of the roof are trimmed to the size required, and the roof boarding is fixed, upon which is spiked or screwed a rough wood curb dovetailed at angles, which should be at least 6 inches deep and flush with trimming and trimmed rafters; these are then cased with wrought and beaded linings, rebated to receive plastering or boarding. In most cases, the wrought and beaded curb lining is made at least 2 inches in thickness, grooved and tongued at angles, and the rough curb is omitted. This construction is more economical, is sufficiently rigid, and renders it possible to reduce the width of styles, top and bottom rails. The curb is covered with lead on the outside to render it watertight.

Fig. 648.

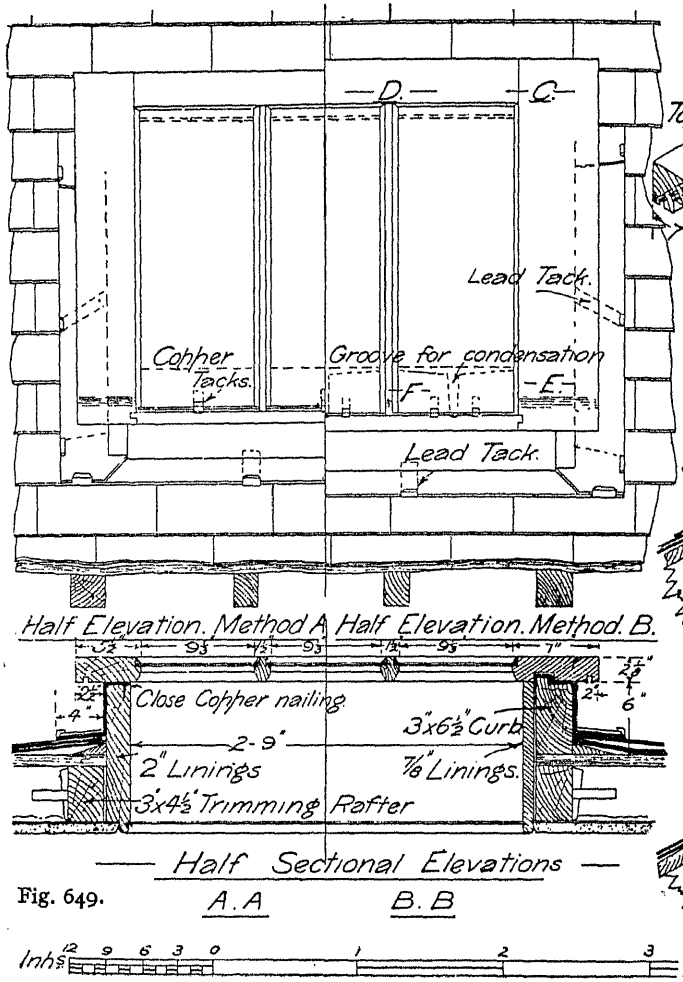


Fig. 649.

Fig. 652.

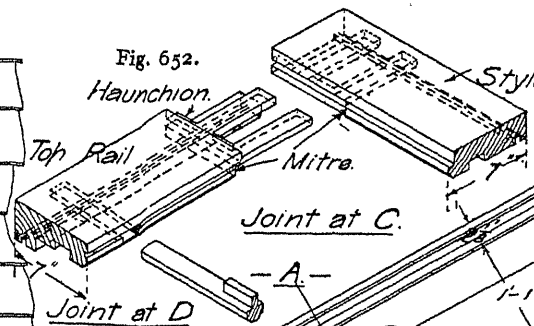


Fig. 650.

Fig. 651.

Fig. 653.

Fig. 654.

Figs. 648—654.

[*Between pages* 844 *and* 845.

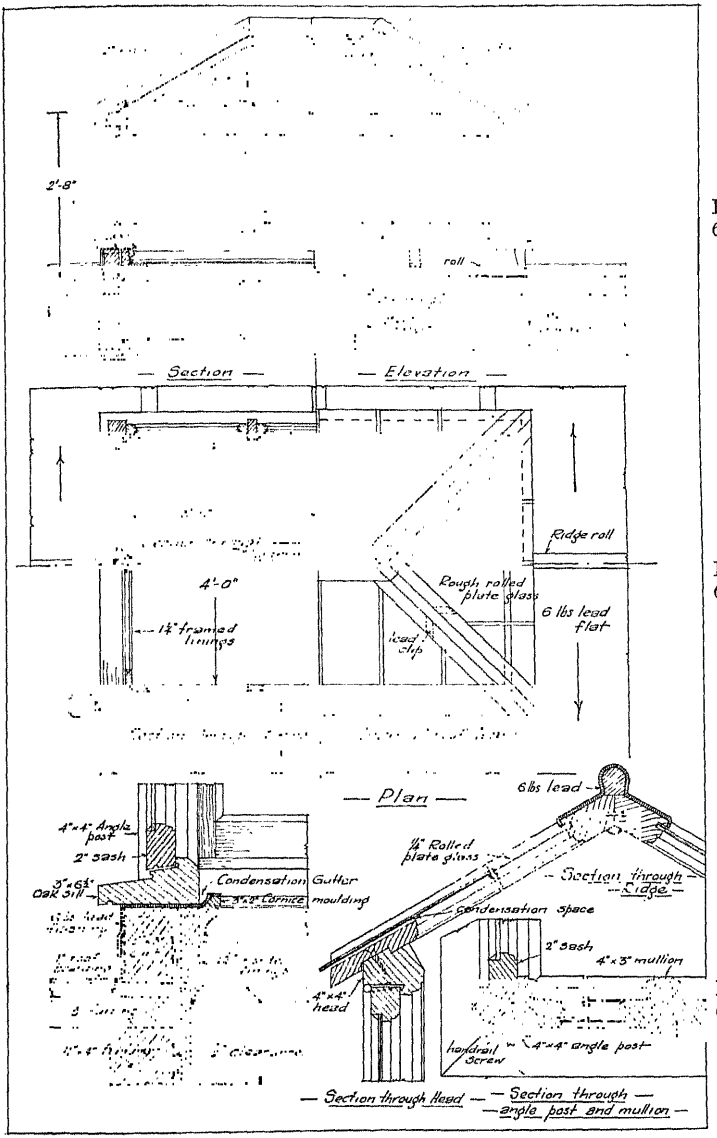


Fig. 655.

Fig. 656.

Fig. 657.

Fig. 658.

Fig. 659.

Fig. 661.

Fig. 660.

Figs. 655—661.

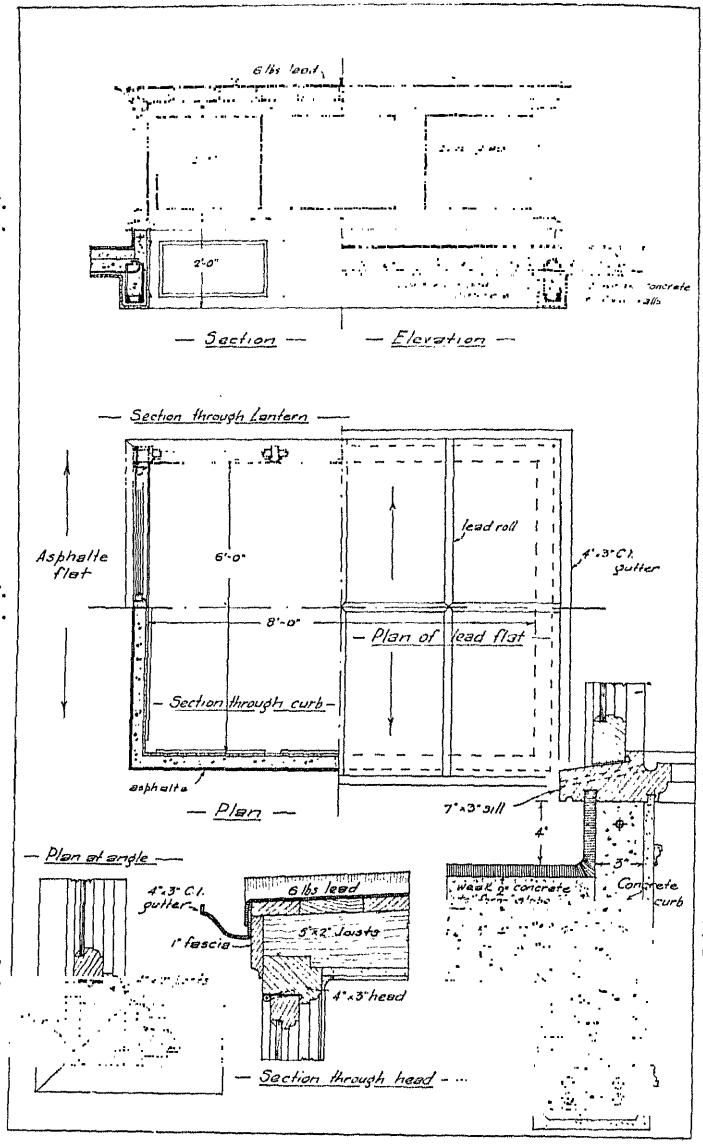


Fig. 662.

Fig. 663.

Fig. 664.

Fig. 665.

Fig. 666.

Fig. 668.

Fig. 667.

Figs. 662—668.

The skylight is placed on the top of the curb to which it is hung, extending over it for at least 2 inches on every side, and being throated on the underside on all four edges. The underside of the top edge often has a fillet about 1 inch in thickness fixed to it, projecting below the top edge of the curb, as an extra precaution against water finding its way in at that part. The sash projects over the curb on its inner edge about $\frac{3}{4}$ inch on the sides and top rail, and about $1\frac{3}{4}$ inches on the bottom rail, the extra width here being required for fixing the apparatus for opening the sash. The sash is constructed slightly differently from other sashes, the variations being as follows: The top rail is grooved instead of being rebated for the glass. The bottom rail is made of a less thickness than the remainder of the sash, the upper surface of the rail being level with the rebate to allow the glass to run over it; the top upper edge of the bottom rail is rebated to form a gutter to collect condensed vapour, this being carried off by transverse grooves, as shown in Fig. 654. As an alternative to this, the whole surface of the rail is sometimes kept below the rebate, as shown in Fig. 653.

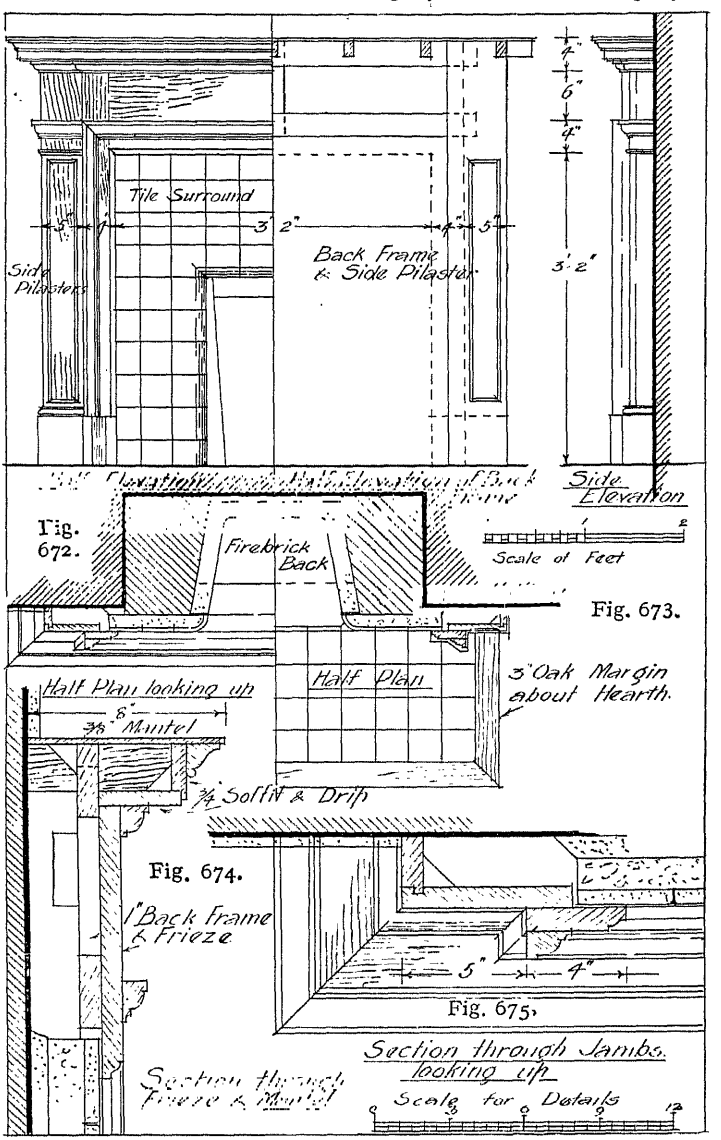
Lantern Lights.—These are an improved form of skylight; they are used chiefly for flat roofs, and also on the ridges of pitched roofs. They consist of a box-like arrangement, being square, rectangular, or polygonal in plan. They usually have the sides vertical, as shown in Fig. 655, but sometimes they are arranged in an inclined position. The following is the method of building an ordinary lantern on a flat roof: The roof timbers are cut and trimmed about the required opening; the boarding is then laid on the roof. A curb is then constructed about the opening, this being at least 6 inches in height; the curb is usually prepared from 6″ × 4″ fir bevel halved or dovetailed at the angles, or if the roof is of concrete and steel the curb may be arranged as shown in Fig. 668; the inside linings, which may be plain or framed, are now fixed, the top of the linings being level with the top of the curb. A moulding is fixed about the top of the lining, having a groove taken out of its top back edge. The lead work of the roof is laid, being turned up about the curb; a flashing is placed

on the curb, being nailed to the edge of the moulding mentioned, and dressed down over the curb. Small grooves are taken out of the curb at about 3 feet intervals, into which the lead flashing is dressed, these forming ducts to carry off the condensed vapour ; or in the case of a steel curb, the ducts are cut out of the underside of the sill. An alternative method of carrying off the condensation is to work a moulding on the inside of the sill, having a gutter worked on the top, with ducts bored through the sill, as shown in Fig. 668. The sides of the lantern are now placed on the curb ; they consist of a rectangular light of four solid frames, each angle post being common to two frames, or as an alternative method four frames may be constructed which are connected at their angles by means of handrail screws through the head and sill, as shown in Fig. 661 ; sometimes intermediate mullions are added ; the sill is usually of oak, being double sunk and throated. It is made to project at least 1½ inches in front of the curb ; the inside of the lantern is kept flush with the inside linings, as shown in Fig. 659. This method is preferable for large frames, being more portable and easier to hoist in position without the fear of racking.

There are three general methods of roofing lantern lights. First : They may be covered with four sashes constructed similarly to skylight sashes, and mitred together at their angles, and forming a hipped roof, the joint being covered with a roll, and after glazing the top rail is covered with lead, as shown in Fig. 658. Secondly : They may have a hipped roof, constructed of moulded bars. The hip, ridge, and other bars will be of different sections, owing to their varying inclinations. Thirdly : They may be covered with a flat roof, in which case light joists having a fall in two directions are placed parallel to two of the sides ; they are boarded and covered with lead. A cast-iron gutter is placed about the roof, as shown in Figs. 662 to 668, and a downpipe at one angle to carry off the rain.

Finishings to Fireplaces.—Fireplaces are frequently finished with a wood surround. This admits of an infinite variety of designs, but in most cases the surround consists

Fig. 669. Fig. 670. Fig. 671.



Figs 669—675.

of a frame with a moulded architrave about the opening and forming a support for a mantelshelf. The immediate surround of the fire opening is usually formed of tile, marble, or some incombustible material, in order to keep the woodwork as remote as possible from the fire. In any case, the high temperature in the immediate vicinity of the fire renders it necessary to select thoroughly seasoned timber for the woodwork. Figs. 669 to 675 show an example with side pilasters, an architrave superimposed, a frieze, and a cornice, which latter forms the mantelshelf. The construction consists of a back frame to give the necessary projection. On the face is fixed a front frame moulded on its inner edge. A narrow moulding is planted on to form the architrave. The cornice or mantel is fully shown in Fig. 674. The cap mould and necking of the pilaster, the base and baseblock are planted on, and a moulded sunk panel in the pilaster completes the fitment. Where flues are formed in the breast the wood frame should be fixed with metal holdfasts cemented in the joints of the brickwork, as no wood plugs are permissible or desirable within twelve inches of any flue.

Shop Fronts.—The ground storeys of commercial buildings arranged for the display of goods admit of an infinite variety of designs, but the same underlying principles govern all types. The main consideration from the merchant's point of view is to have as large an area as possible for display, unobstructed by large pillars or other constructional details. The design of fronts varies according to the nature of the goods to be exhibited, but the constructional details mainly depend upon whether protection by means of shutters is required. Where the goods are large or of secondary value, shutters may be dispensed with, and the frame to contain the glass becomes merely an enlarged sash frame of wood, cast iron, bronze, or wood covered with pressed sheet bronze fitted into the reveals of the brick or stone piers which constitute the surround of the shop. In the old types of shop fronts, wood shutters were employed. These were usually framed and panelled, and rebated to each other along their side joints. They were fitted into a groove along their top edges and were

Fig. 676.

Fig. 677.

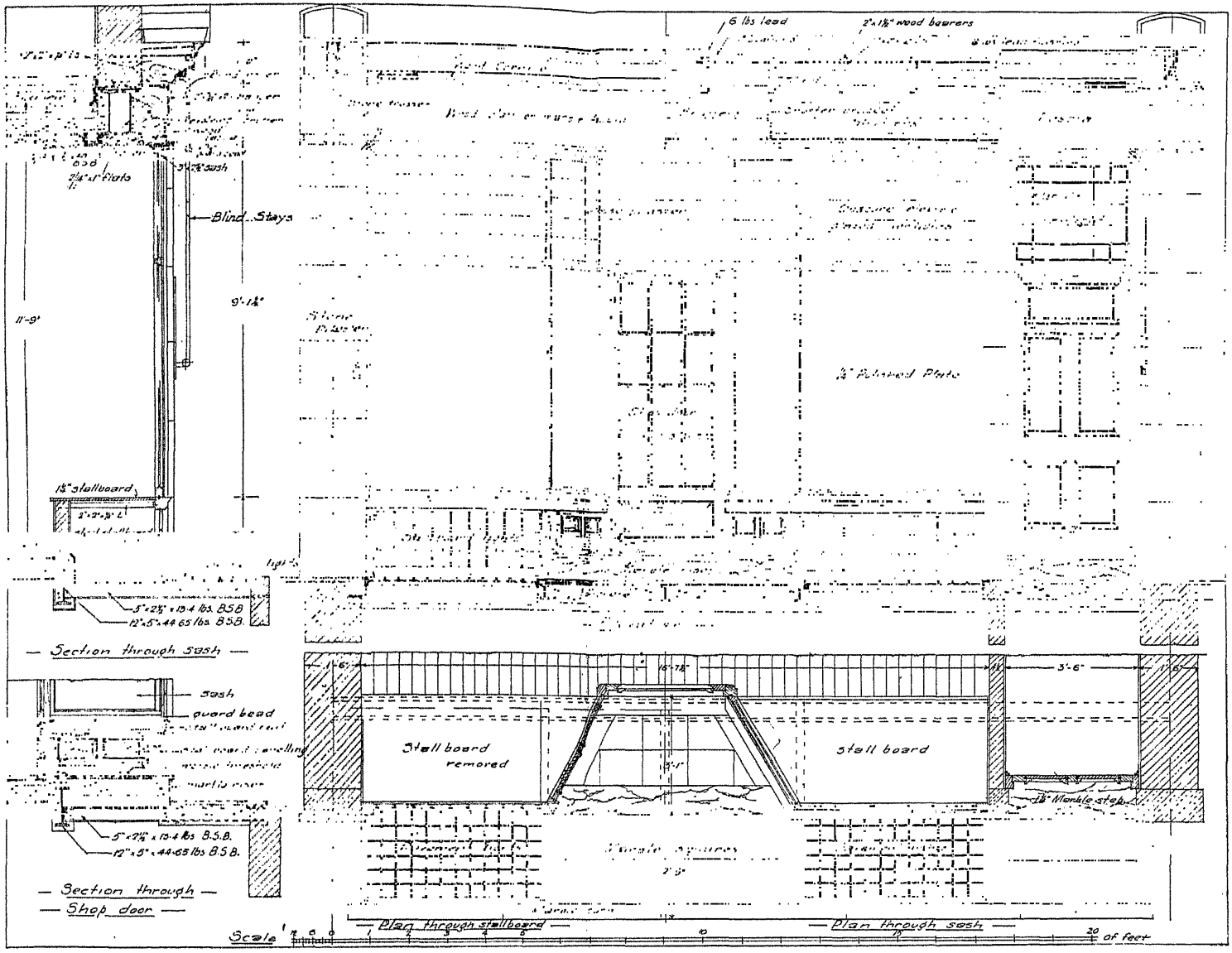


Fig. 678.

Fig. 679.

Figs. 676—679.

[Between pages 850 and 851.

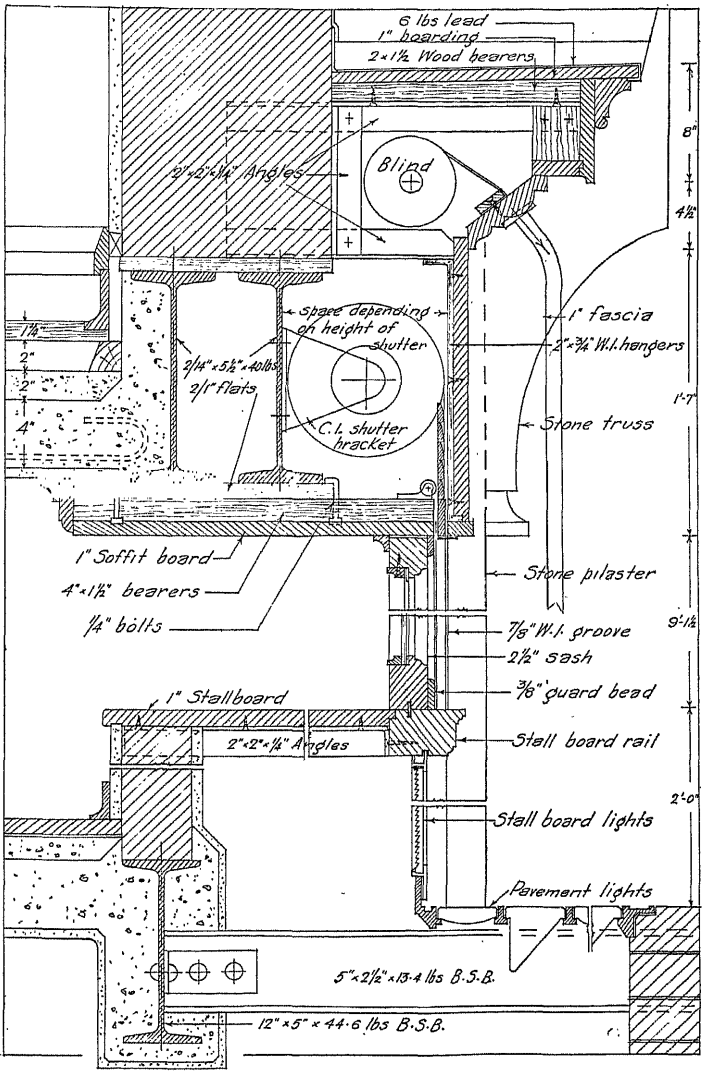

6 lbs lead
1" boarding
2×1½ Wood bearers
2"×2"×¼" Angles
Blind
Space depending on height of shutter
C.I. shutter bracket
2¼"×5½"×40lbs 3/r flats
1" fascia
2"×¾" W.I. hangers
Stone truss
1" Soffit board
4"×1½" bearers
¼" bolts
Stone pilaster
⅞" W.I. groove
2½" sash
⅜" guard bead
1" Stallboard
2"×2"×¼" Angles
Stall board rail
Stall board lights
Pavement lights
5"×2½"×13·4 lbs B.S.B.
12"×5"×44·6 lbs B.S.B.
8"
4½"
1'-7"
9'-1½"
2'-0"
1½"
2"
2"
4"

Fig. 680.

secured by a stout iron bar placed at about one-third of their height. This was let into a mortise in the side pilasters at one end, and the other end was secured by a bolt passing through the bar, the shutter and the frame, and secured by a wing nut on the inside. This type is now obsolete, being replaced by revolving shutters. When the shop is open the shutter is concealed in a boxing provided to receive it and formed in the fascia above the shop front, see Figs. 676 and 680. Where the fascia or architrave is of stone, the boxing is usually fitted to a subfascia immediately below the stone architrave. When the shop is closed, the shutter is drawn down, the vertical edges sliding in grooves formed in the side pilasters provided to receive them. Figs. 682 and 685 show the principle of fixing the shutter in a subfrieze below a stone entablature.

Nearly all shop fronts are provided with sunblinds formed of stout canvas, secured on the upper edges to encased springs similar to those on the shutters. The blind box is fitted either in the wood entablature or is fitted below the stone entablature. Figs. 681 to 687 show the blind box fitted below a stone architrave.

The advent of prismatic pavement lights renders it possible to turn what in the past was a dark cellar or basement storey fit only for the storage of goods into valuable day-lit shopping areas. To obtain the maximum benefit from these prismatic lights, the shop floor should be raised sufficiently above the pavement level to allow the refracted rays of light emerging from the pavement lights an unobstructed passage into the interior.

Fig. 680 shows an example with revolving shutters. and a sunblind arranged in a wood entablature

The fascia and cornice is the first part to be fixed, being secured in the older types to bracketing fixed into the face of wall, about the bressummer. In more modern methods light steel angles are employed. These are stronger and do not shrink on drying like wood ; they make a more rigid job and do not take up so much room as the wood brackets (see Fig. 680). Three light angles bolted form the supports of the cornice. These are built into the wall and are placed at about 2 feet centres. A wood bracket bolted to the outer end of the upper part of the steel forms a fixing

for the crowning mouldings. A strong wood bracket is bolted to these sections where required for the supports for the sunblind. The bed mould of the cornice forms the batten for the lower edge of the sunblind.

The revolving shutters may be supported on cast-iron brackets bolted to the girders, or they may be fixed on to strong wood brackets wedged between the flanges of the girders, at the extremities of the shutters, usually at about 7 feet centres (see Fig. 680).

The fascia, usually made from straight-grained mahogany, is removable to give access to the shutter. It is screwed to the wood brackets before mentioned or to light steel angles. At these points wood brackets are fixed to the lower flange of the girder (see Fig. 680) and to these the wood soffit is fixed. Laminboard, which can be obtained the full width required, without a joint, is now frequently employed for these wide surfaces. To the back of the fascia steel bars $1\frac{1}{2}'' \times \frac{3}{8}''$ slot screwed are fixed; these are bolted to the upper bracketing as shown in Fig. 680.

The shutter is fixed at one edge to a hollow cylinder, containing a strong spring fixed to a central spindle projecting at each end of the cylinder; the projecting parts are made square, being placed in a socket screwed to the strong brackets already mentioned; the central spindle does not revolve, only the hollow metal cylinder to which the bands passing through the shutter are fixed; when the shutter is opened the spring is strained, and when closed it tends to regain its normal condition, thus making it easy to lift the weight of the shutter. The revolving shutters slide along vertical iron grooves, screwed into the side of pilasters fixed at both sides of the shop front; at intermediate distances between the two outside pilasters, upright members with grooves on their two edges, termed loose pilasters, as shown in Fig. 677, fitted with studs and plates in their upper and bolts at their lower ends, are placed directly below the bracket supporting the shutters to secure the free edge of the shutters when open. When the shutters are closed these pieces are taken down.

The stall board framing is now fixed in the correct position below the fascia. On the top of the stall board framing a sill is fixed, as shown in Figs. 676 and 680. In

many cases where there is a basement, a sash is substituted for the framing ; steel sashes with prismatic lights are now usually employed where the maximum daylight is required in the basement. Figs. 676 to 680 show examples with steel sashes and prismatic pavement lights. The sashes are then erected on the sill, between which and the cover-board (that is the soffit below the girder) it is accurately fitted and fixed. About the sash a thin fillet, rounded on one edge, termed a guard bead, is fixed ; this acts as a scribing fillet to make a good finish and close joint about the sash, and also to prevent the shutters rubbing against the sash.

Figs. 676 to 680 show complete working drawings for a double-fronted shop front with a wood entablature.

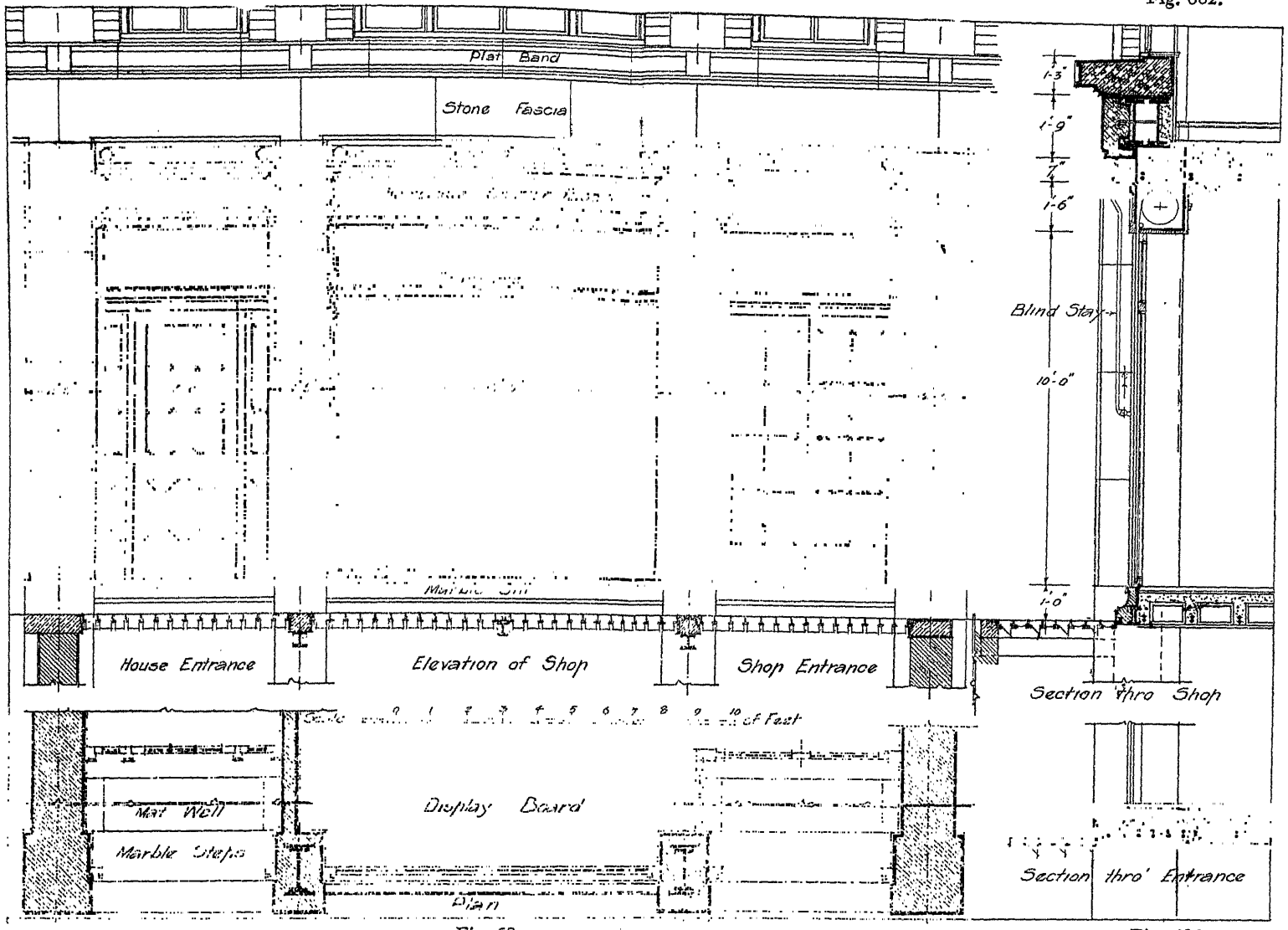
Figs. 681 to 689 show a case with revolving shutters and a sunblind fixed within stone surrounds. The sash is supported upon a stone or marble sill which raises the sash the necessary height above the pavement. The sash is bedded in white lead on the stone sill, and is provided with an iron tongue to prevent a through joint. The sides of the sash are fitted into shallow reveals formed in the sides of the stone piers. Where there is a shutter the grooves are usually formed as shown in Fig. 687 in the styles of the sash. The back and soffit of the blind box are formed of wood fixed to steel straps or hangers suspended from the floor above. The front edge of the soffit and hanger rest on the top rail of the sash. The brackets which support the shutter are bolted up into the floor, or secured to the flanges of the steel bressummer. The front of the shutter boxing and the blind boxing are fixed to steel hangers suspended from the floor or from the bressummer.

The sash and other woodwork may be formed from hard wood polished, but it is now customary to use sections covered with sheet bronze.

The large sheets of plate glass are fixed in the rebates with beads. The glass may be inserted into the frames either from the inside or from the outside, the latter method being common practice when the sheets are very large. The difficulty and risk in handling is minimised when the glass is inserted from the outside.

Fig. 681.

Fig. 682.



Figs. 683 and 684.

Fig. 685.

Fig. 686.

Figs. 681—686.

Between pages 854 *and* 855.

Fig. 687.

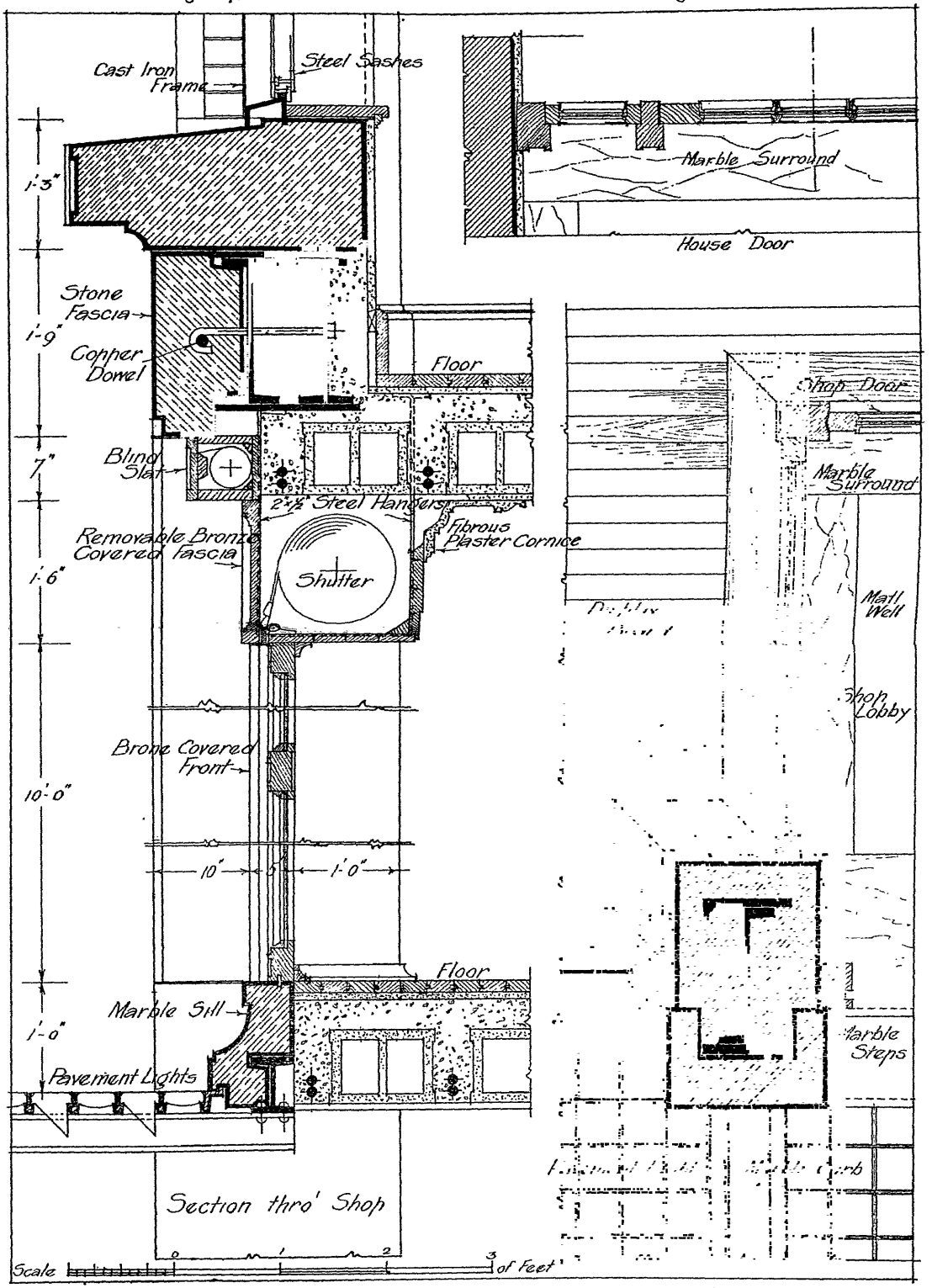


Fig. 688.

Fig. 689.

Figs. 687—689.

[Between pages 854 and 855.

Projection of Shop Fronts.—The London Building Act, 1939, para. 131, (*d*), states :—

(i) if the shop front is situate in a street of a width of not more than 30 feet the shop front (including any pier in connection therewith) shall not project more than 5 inches and the cornice and corbel thereof shall not project more than 13 inches from the building ;

(ii) if the shop front is situate in a street of a width of more than 30 feet the shop front (including any pier in connection therewith) shall not project more than 10 inches and the cornice and corbel thereof shall not project more than 18 inches from the building ;

(iii) the cornice and corbel of the shop front may project to an extent not greater than is mentioned in the foregoing paragraphs of this proviso over the roadway of the street or over any ground left open.

Woodwork of Shop Fronts.—The London County Council Byelaws, para. 143, state :—

(3) No part of any shop front shall be fixed higher than 25 feet above the level of the ground immediately in front of the shop.

(4) No part of any shop front (including the cornice or fascia thereof) shall be fixed nearer to the centre of a party wall or (where there is no party wall) to the external wall of adjoining premises than 4 inches or if such shop front projects more than 4 inches from the front of the building to which it belongs nearer than the amount of such projection unless it be separated from such adjoining premises by a pier extending 1 inch in advance of every part of such shop front and a corbel projecting 1 inch in advance of any cornice or fascia thereof. The pier and corbel shall be at least 4 inches wide and shall be between the centre of the party wall or (where there is no party wall) between the external wall of the adjoining premises and the shop front, and shall be constructed of brick stone or other incombustible material. Provided that nothing in this paragraph shall authorize the construction of any part of such pier on or over the public way or any and to be given up to the public way.

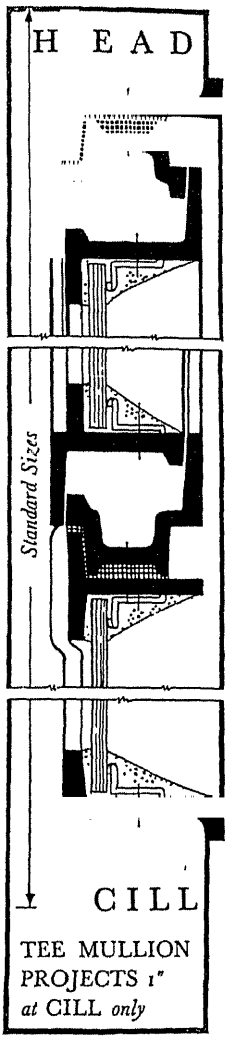
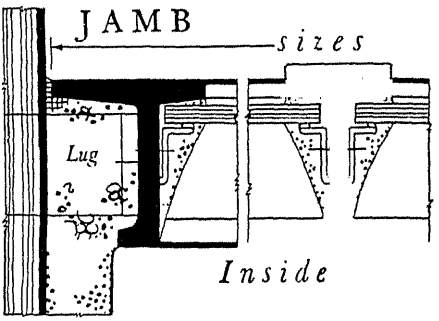


Fig. 690.

Fig. 691.



MULLION COUPLING

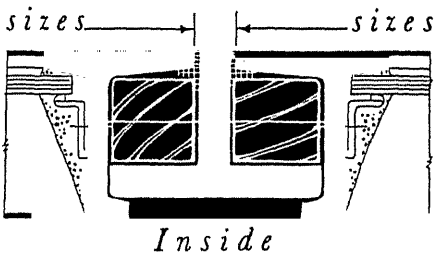


Fig. 692.

Fig 693.

Figs. 690—693.

Substitutes for Joinery Work

Many fitments that were once made exclusively by the joiner are now being made in metal; in such articles as office furniture and fitments the use of pressed metal is now a

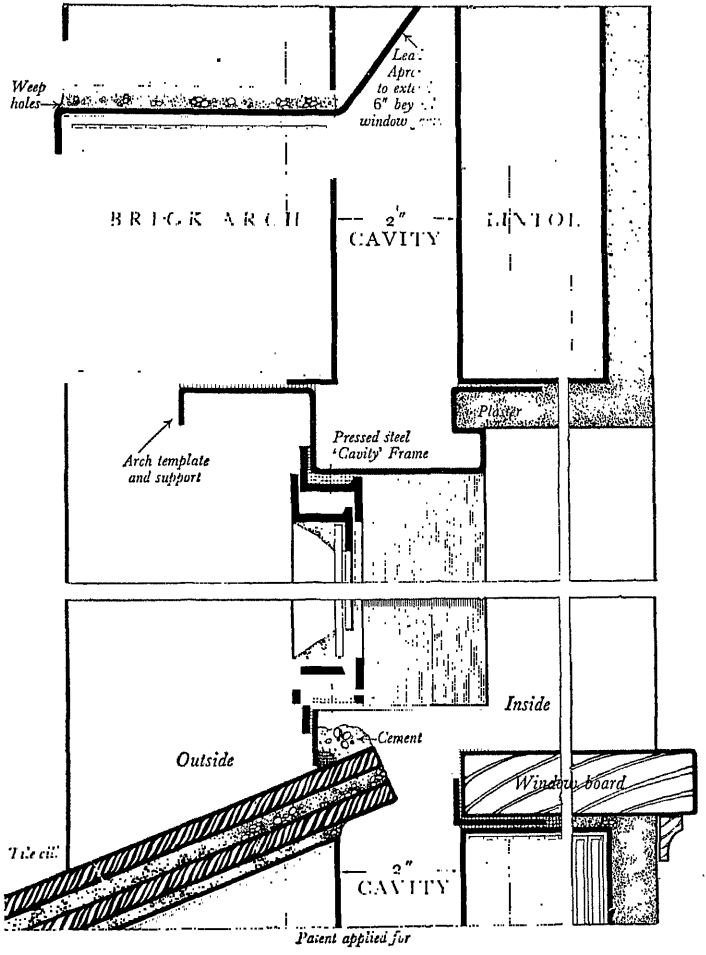


Fig. 694.

rapidly growing practice. In construction, steel and bronze windows, which have been used for many years and are now well past the experimental stages, are being used for all classes of buildings as they offer many advantages over wood for all types of dwellings, industrial buildings, ware-

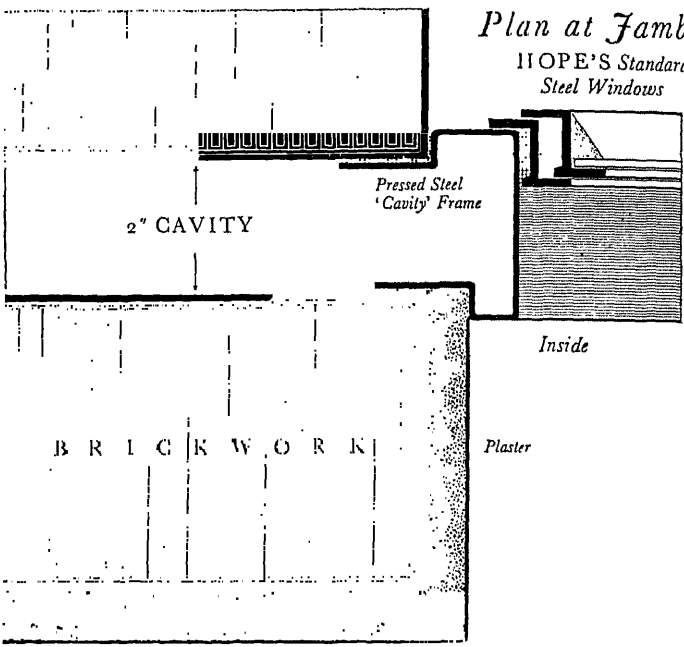


Fig. 695.

houses and engineering shops where maximum light and controlled ventilation is essential.

1. Steel sashes are fireproof and hygienic.

2. Steel, unlike wood, does not swell or shrink with variations in the humidity of the atmosphere, and the opening casements and ventilators are unaffected as would be the case with wood sashes.

3. The obstructions to light are much less with steel than with wood.

4. In metal sashes made by firms of repute, the surfaces between the sash and the frame are made to come into perfect contact, and there are usually two of such surfaces, with a large space between that prevents capillarity, thus draughts are prevented, and rain driving through is an impossibility.

With wood sashes there must always be a clearance, and, although capillarity may be prevented, draughts are always present. There is also an advantage from the economical stand-point, as steel windows and sashes are always supplied with ventilators ready hung and with all fasteners fixed, which saves much time and expense to the builder.

Figs. 690 to 693 show head sill jambs, transomes and mullions for the standard sashes suitable for industrial buildings.

Messrs. Henry Hope & Sons manufacture pressed steel surrounds, which eliminate all woodwork, for use where the extra width of frame is desired. This has the advantage of maintaining the fireproof qualities of a building, and also allows the builder to proceed with the building and to fix the windows when all the rough trades are off the job.

The sills are specially adapted for different external finishes and are specially designed with a groove to receive the window boards or tiles internally (see Fig. 698).

Lanterns. Steel sashes and roof glazing bars also provide an excellent combination for forming all kinds of weather-resisting lanterns, skylights and roof coverings. Fig. 699 shows a general view and Figs. 700 to 702 show details of the head sill and ridges for a steel lantern.

Fig. 695 shows steel sub-frames for standard windows fixed into cavity walls ; the heads are made to project to form a support upon which to build upright brick lintels or the so-called soldier arches (see Fig. 694). A special surround is also supplied for the formation of double noise-excluding casements, which supplies a much-felt want in offices and residences in noisy thoroughfares (see Fig. 698). Double windows have also the advantage of being heat retaining in the winter season.

All windows are painted a special red oxide priming

coat before they leave the works, but it is of paramount importance if steel windows are to be kept free from corrosion that they should be well painted after they have been fixed in the openings, and unless this painting is

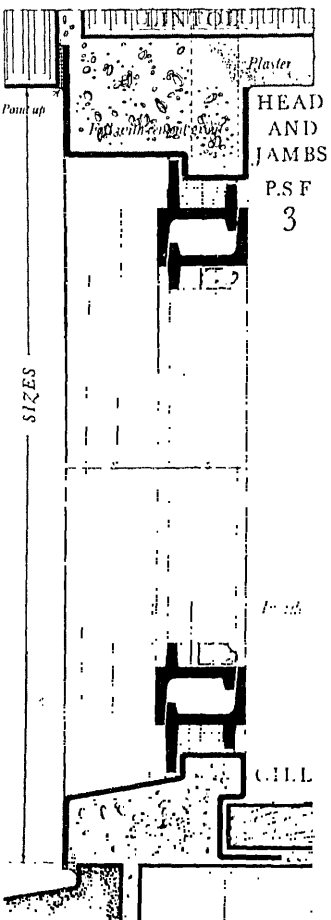
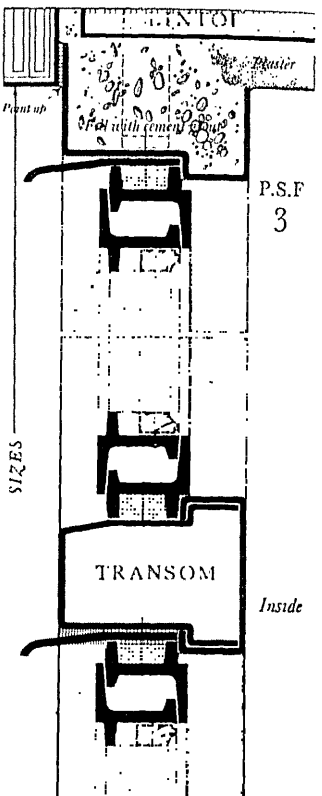


Fig. 696.



Windows can be fixed into steel frame rebates as shown above The remaining three details show the windows lining with inside face of sub-frame, as this is the most convenient position for the fixing of cable gear for operating transom lights

Fig. 697.

Figs. 696—697.

well done with first-rate material corrosion is likely to occur.

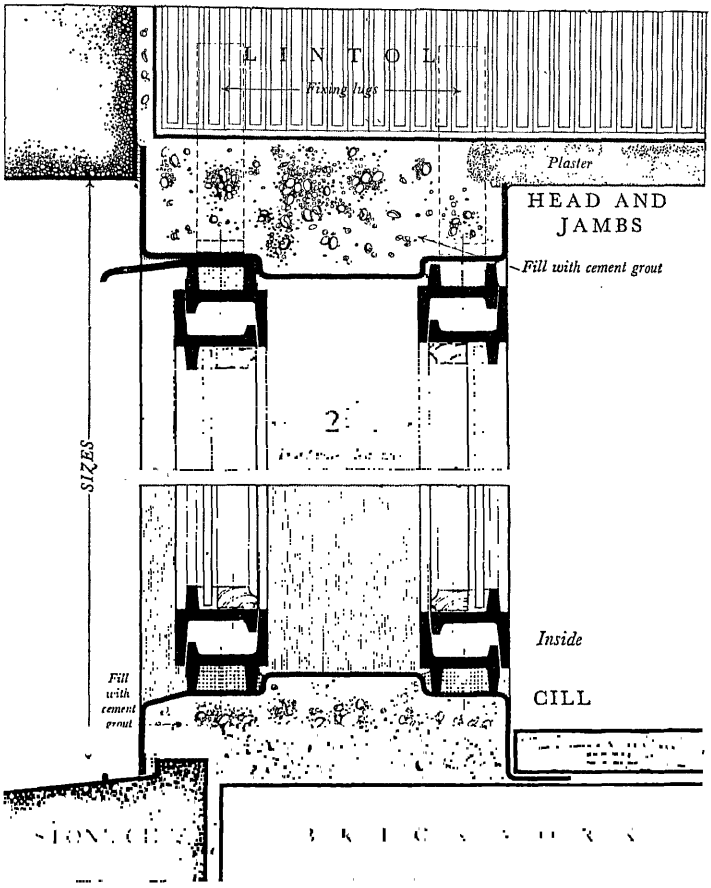


Fig. 698.

After the windows are fixed they should be cleaned of all dirt and foreign matter and painted one coat of either

genuine white lead or a genuine zinc oxide paint. After glazing, two more coats should be applied when the putty has had time to harden. The quality of these final coats of paint is very important and should not contain too much thinners or dryers. If this painting is conscientiously done and the windows given reasonable attention at, say, intervals of every four years, they will last indefinitely.

Sherardizing is also recommended for the protection of steel windows. Sherardizing is the name of a process for applying zinc to the surface of articles made of iron or steel to prevent rust. The process is quite distinct from most

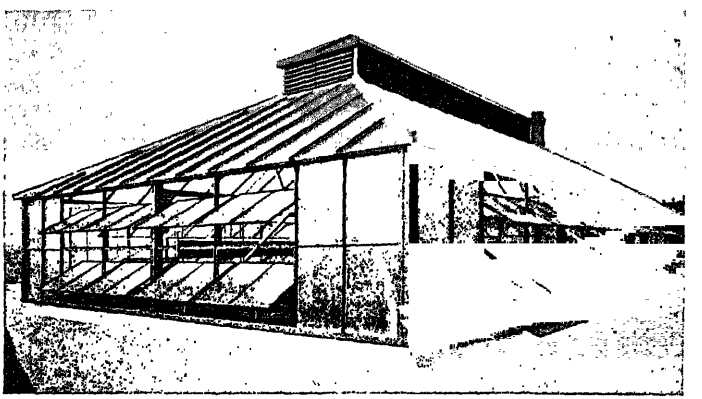


Fig. 699.

other methods of treating the surface of metal articles in one important detail, this is, that the zinc protection form is actually an alloy of zinc with the surface of the article treated.

Door Trim. Steel door frames and linings have been standard practice in America for a considerable time. In Britain they have not been used until recently. Messrs Hope & Sons have recently installed plant for the production of pressed steel door linings, which are now on the market. They supply a need and because of their durability

and fire- and vermin-proof qualities, bid fair to become as widely used as the applications of the steel sash.

The frames are made in a number of designs to suit any

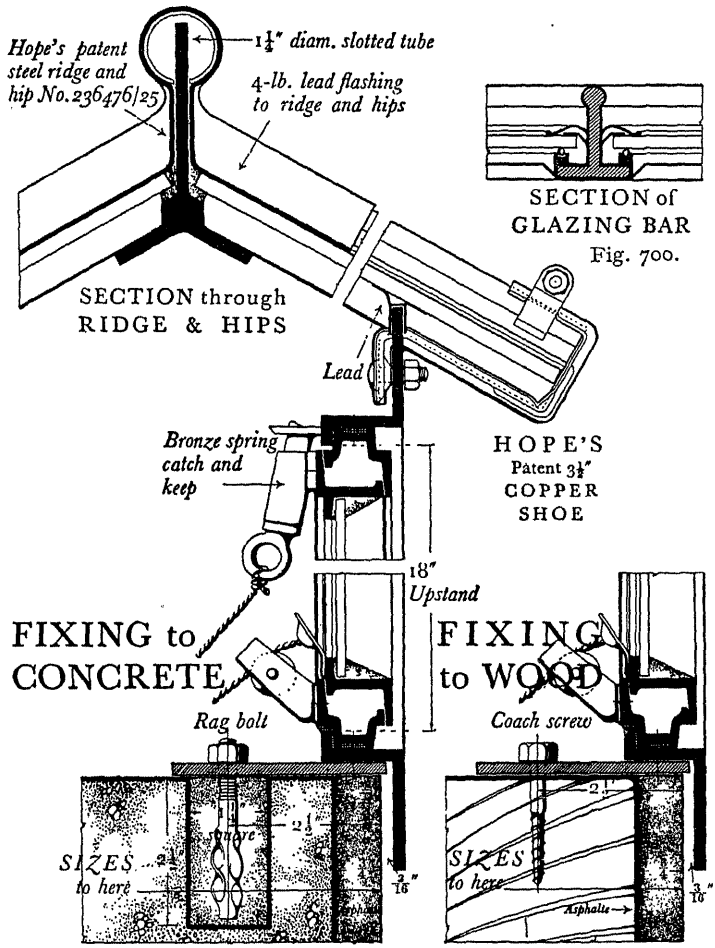


Fig. 701. Fig. 702.

Figs. 700—702.

type or thickness of wall or partition, also any type of door. They are constructed of best quality 16-gauge

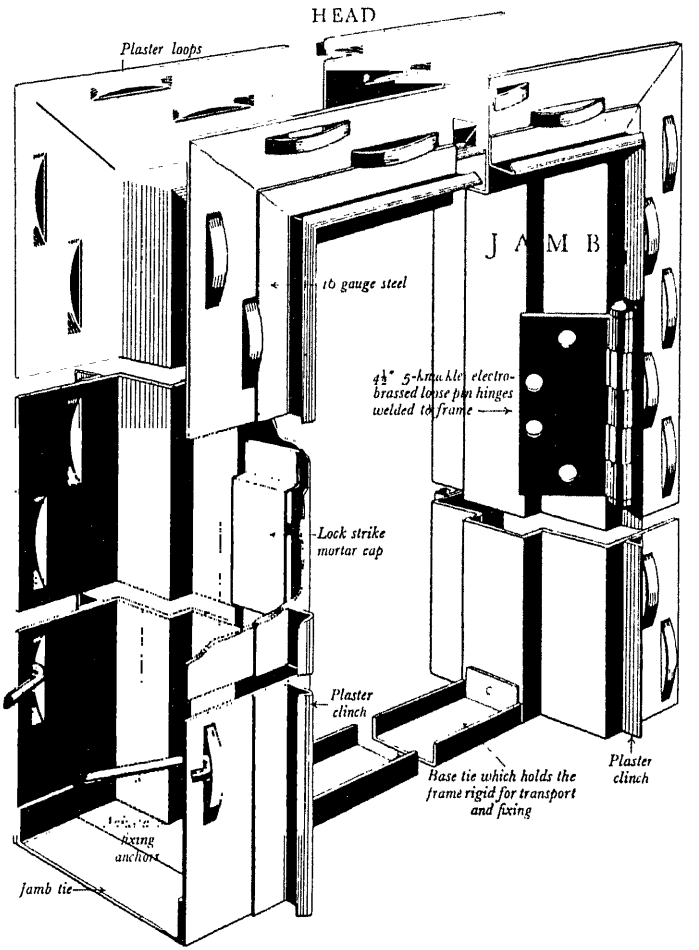


Fig. 703.

British Steel Sheets, cold rolled, close annealed and hydraulically flattened.

Figs. 703 and 704 show two types of these frames in perspective.

Frames are welded at the corners and provided with a

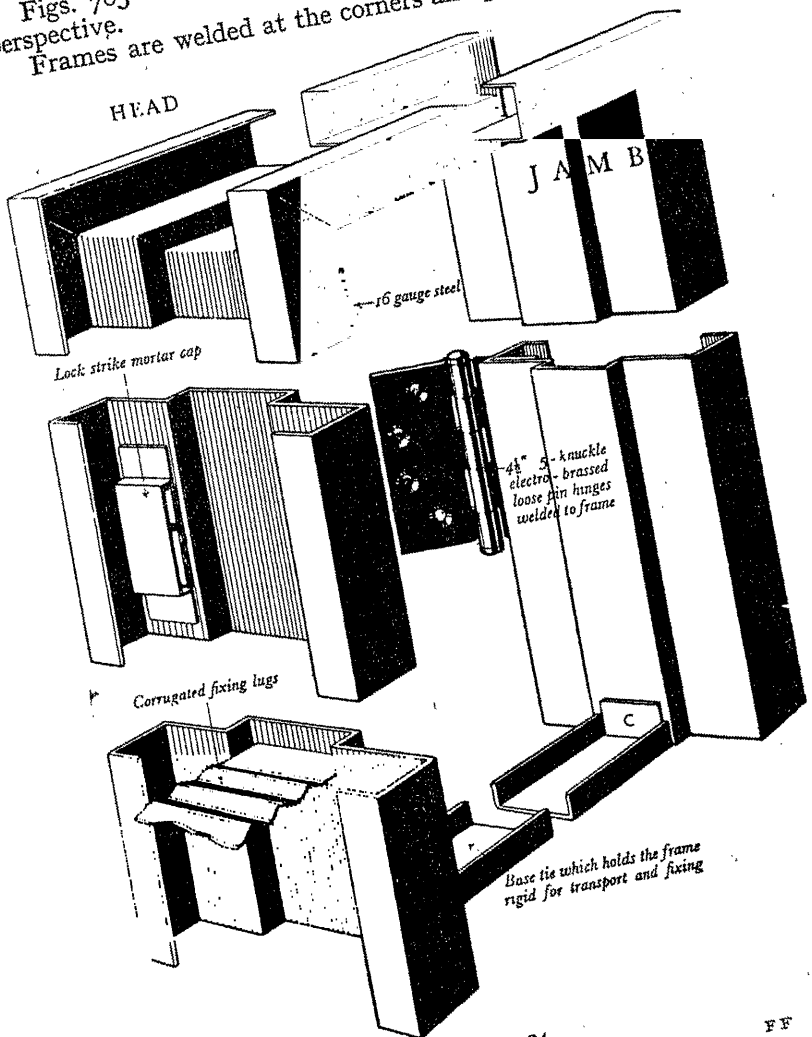


Fig. 704.

FF

3.C.

base tie for rigidity during transport and fixing. After fixing, the base tie can be removed if desired, or left in

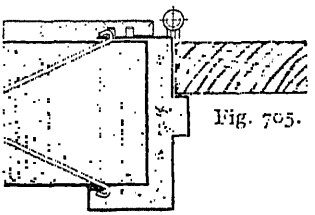


Fig. 705.

Type A2

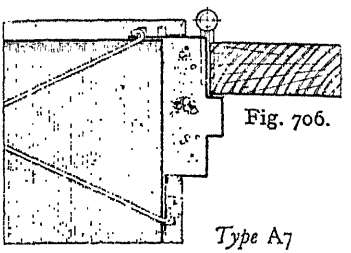


Fig. 706.

Type A7

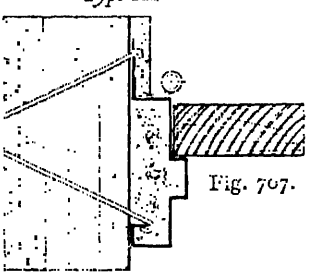


Fig. 707.

Type A5

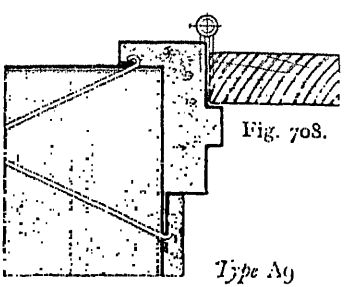


Fig. 708.

Type A9

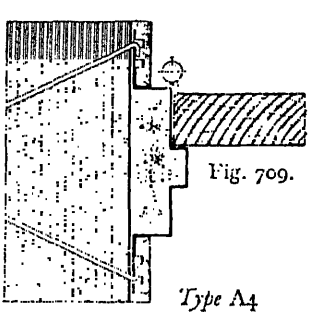


Fig. 709.

Type A4

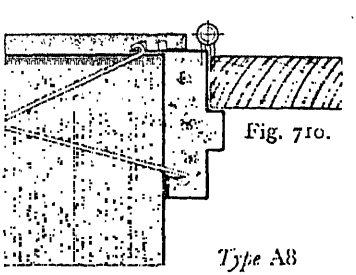


Fig. 710.

Type A8

Figs. 705—710.

place and the floor carried over. All frames are painted with red oxide paint, stoved on at 250° Fahr. They are vermin-proof, fire-proof and rot-proof.

The hinges are of great strength and fine quality; they are welded to the steel frames and cannot possibly come

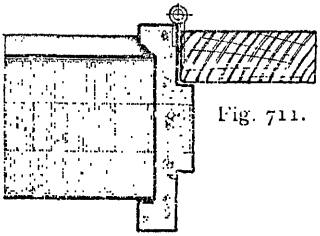


Fig. 711.

Type B2

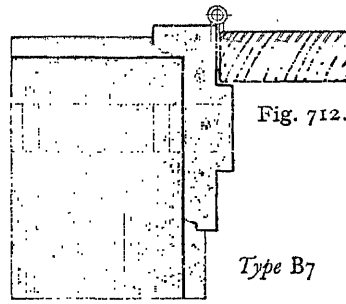


Fig. 712.

Type B7

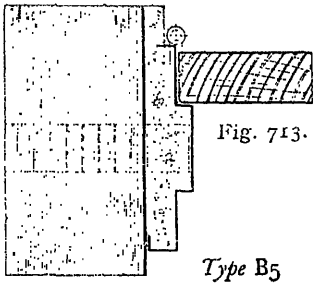


Fig. 713.

Type B5

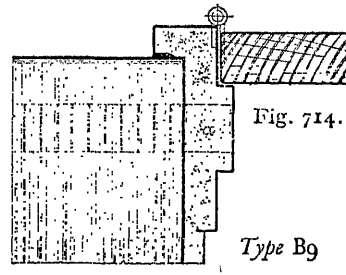


Fig. 714.

Type B9

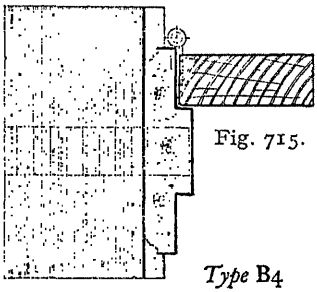


Fig. 715.

Type B4

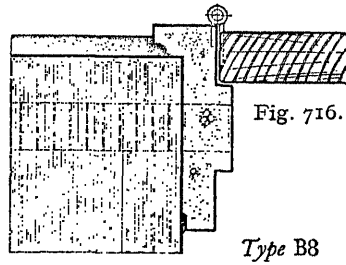


Fig. 716.

Type B8

Figs. 711—716.

loose or allow the door to drop. Loose pins are fitted so that a door can be removed when required in a few seconds without the use of tools.

Fixing is extremely simple. The steel frame being set plumb in position, the bricklayer should build his brick wall or slab partition inside the jambs, building in the fixing lugs or anchors as the work proceeds, flushing the hollow space with mortar or cement at each course, so as to make a solid job. Owing to the heavy gauge steel employed, and the stout steel base tie sent with every frame, they are rigid when placed into position, and when once fixed, maintain their shape.

The finishing coat of plaster is applied to the wall in the usual way. For grade "B" frames, the plaster should be well flushed into cavity between frame and brickwork, and neatly struck down the returned face of metal frame as shown on details.

Grade "A" frames are perforated on the returned edges to form a plaster key, and a steel plaster clinch is provided which forms a first-class non-cracking finish against the door jamb, as shown on p. 867.

It should be noted that the provision of hinges on the steel frames reduces the cost of carpenters' time in hanging the doors.

Figs. 705 to 716 show the applications of these frames to different types of jambs.

The illustrations of steel sashes, lanterns and door frames have been supplied by Messrs. Henry Hope & Sons, of Smethwick, Birmingham, one of the pioneer and leading firms for this type of work.

CHAPTER XXIV

STAIRS

Materials.—Stairs are made in stone, concrete, ferro-concrete, brick, iron, and timber; all but the latter two have been already treated.

Iron Stairs.—Iron stairs are used for internal and external constructions, where there is a minimum of space to be occupied, and where the least obstruction to light and air, and a measure of fire-resistance and economy are chiefly required, such as where the internal plan space is small, spiral steps are used, and for external stairs to buildings; they are not combustible, but they are slippery when worn, and are not much used for other purposes.

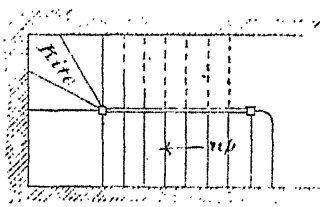
Planning of Stairs.—Buildings should be designed for and provided with convenient staircases, and the planning of the stairs considered as of primary, and not of secondary, importance.

Design of Stairs.—Properly designed stairs should (*a*) be well lighted and ventilated directly from the exterior; (*b*) have the approaches convenient and spacious; (*c*) have the headroom in no case less than 7 feet measured vertically; (*d*) have a clear width between the strings of the straight portion of flights kept at all turns, landings, and approaches; (*e*) have the stairs of a convenient and easy pitch; (*f*) not have a landing between two adjacent flights, the centre lines of which are in one straight line, with a length less than the width of the adjoining stairs; (*g*) have winders (if any at all) at the bottom and not at the top of a flight; (*h*) have not less than four steps in each flight.

The non-compliance with these conditions has resulted in numerous accidents.

Technical Terms.—Stairs.—Timber stairs or steps consist of a number of wooden blocks or casings fitted into or resting upon inclined beams called strings and carriages, which distribute the load upon the main members of adjacent floors.

The trimming of floor joists to form well holes has been shown in the chapter on Floors, *Elementary Course.*



DOG-LEGGED STAIR.

Fig. 717.

Staircase.—The chamber containing the stairs is usually known as the Staircase.

Tread.—The upper surface of a step upon which the foot is placed.

Nosing.—The exposed edge of the tread, usually projecting and moulded.

Riser.—The face of the vertical member directly between the nosing of an upper and the back edge of the lower step.

Fliers.—Steps rectangular in plan.

Winders.—Steps tapering in plan. Those fitting into a wall angle, as shown in Fig. 717, are termed kite winders.

Going.—The horizontal distance between two riser faces.

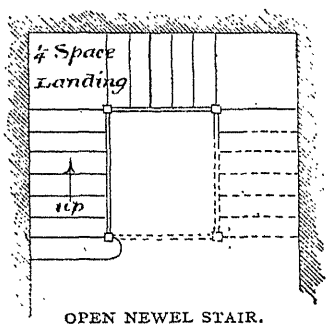
Rise.—The vertical height between two tread faces.

Flight.—A series of steps without a landing.

Landing.—The level platform at the top of a flight between floors.

Half-space Landing.—A rectangular landing extending across the widths of two flights and against one edge, of which both flights abut, and on which a half turn is made.

Quarter-space Landing.—A rectangular landing, the breadth and length of which are of the dimensions of the two abutting flights, as shown in Fig. 718, and on which a quarter turn is made.



OPEN NEWEL STAIR.

Fig. 718.

Line of Nosings.—An imaginary line parallel to the strings and tangential to the nosings. It is useful in the construction of handrails, giving the line with which the under-surface of the handrail should coincide.

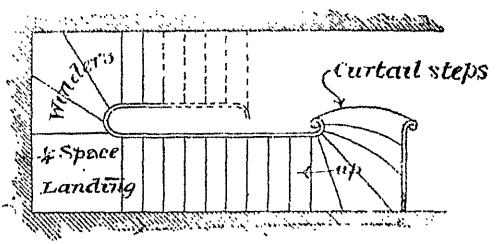
Newels.—Posts forming the junction of flights of stairs with landings or other flats.

If the outline plan of newel stairs enclose a space or solid they are known as open or solid newels respectively.

Straight Stair.—Flight or flights of parallel fliers that may be seen from top to bottom, the centres of which are in the same straight line.

Open Newel.—Stairs, the turns of which have newels and enclose a well.

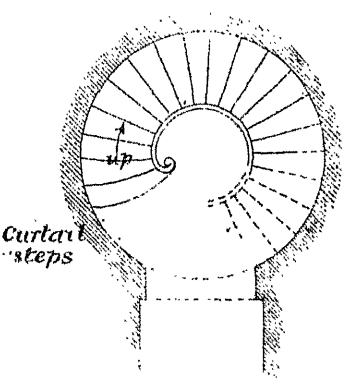
Dog-legged Stair.—A flight of stairs with abrupt angular turns, usually about a newel, and often with winders and landing. The name is given owing to its supposed resemblance to a dog's hind leg, as shown in Fig. 717.



GEOMETRICAL STAIR.

Fig. 719.

Geometrical Stair.—Stairs having continuous strings or handrail, and usually compassing a well. Typical examples of those rectangular and circular in plan are shown in Figs. 719 and 720.



GEOMETRICAL STAIR.

Fig. 720.

Solid Newel.—Stairs radiating from a solid central newel, circular or rectangular in section.

Handrails.—A rounded or moulded member, as shown in Fig. 737, following generally the contour of the nosing line, the upper surface of which is usually 3 feet, minus half-a-rise above the nosing line above stairs, and 3 feet above level platforms. These are usually placed over the outer strings, but in wide staircases should be placed on the wall side also.

Ramp.—A plane curve without change of direction ; it occurs in handrails, as shown in Fig. 738, and in those parts of strings over windows and landings.

Swansneck.—A plane continuous curve, such as would be formed by joining a concave with a convex ramp, as shown in Fig. 738.

Iron Core.—The iron band about $\frac{3}{16}$ inch to $\frac{1}{4}$ inch in thickness, used in the handrail of geometrical stairs to strengthen the curved handrail and to which the balusters are fixed. Iron cores are used in all stairs having iron balusters.

Balusters.—Vertical members between the handrail and strings to stiffen the handrail and prevent persons falling through.

Balusters should be tenoned to close strings and handrails, as shown in Fig. 745, and dovetailed to the treads of cut strings, the return moulding of tread being planted, mitred, with returned end and covering the dovetail, as shown in Figs. 751 and 753.

There are usually two balusters for each step of a cut string, as shown in Fig. 751, and in close strings they are placed about 4 inches apart.

Balustrade.—The framed fence formed by strings, handrails, and balusters, as shown in Fig. 743.

Steps.—The bottom step of the lowest flight in a staircase is usually given emphasis, by being made to project beyond the newel, the outer end being formed either as (1) bull-nose step, (2) a rounded step, or (3) a curtail step.

Bull-nose Step.—The bull-nose step is finished as a quarter round. The face of the riser is made to coincide with the centre of the newel into which it is housed and screwed, as shown in Figs. 721 to 723.

Rounded Step.—The rounded step is finished as a half round, the rounded end usually having a diameter of twice the going. The newel in this case is tenoned through the full depth of the step as shown in Figs. 725 to 730.

The Curtail Step.—The curtail step is usually employed with geometrical stairs, and follows in form the plan of the scroll with which the handrail in the lowest flight generally terminates.

The principle in building up these three types is the same. It is desirable that the grain of the riser on the curved end shall be continuous, as it is impossible to bend material of the thickness of the riser through such a quick curve. The following method of construction is employed. The board which forms the riser is reduced in thickness along the portion that is to be bent to a full sixteenth of an inch. A solid block is then formed, being glued up in at least three thicknesses, the grain of each piece running in differing directions. This prevents alteration in form through shrinkage in one direction. The material should be bone dry to reduce shrinkage to a negligible quantity before the block is glued up. It is then cut to the form of the step. The reduced end of the riser board is then steamed to render it more pliable. The free end is secured to the solid block. The block is well glued, and the board is bent round and is drawn tightly over the rounded surface by a pair of folding wedges. The surface of the veneered portion should be well pressed on to the surface of the block with a caul hammer. Finally the block is screwed to the riser. (See Figs. 731 to 734.)

The scotia moulding is prepared from a board. The exact curve is obtained by placing the curved riser on the board and tracing a parallel curve on the same. When the curve is cut the scotia is laid on the riser board and screwed down. The shape of the end of the tread is then obtained in a similar manner.

Figs. 721, 725 and 731 show the elevation and plans of the three types of steps. Figs. 721 to 723 show the construction of the bull-nose step, Figs. 725 to 730 the construction of the rounded step, and Figs. 731 to 734 the construction of the curtail step.

Built-up Steps.—Wooden steps are usually built up of a number of comparatively thin casings, dressed only on the seen faces. Each step takes a bearing on at least two or three carriages, or rough, or dressed strings, the soffit of which is plastered, as shown in Fig. 738, or it may be boarded.

Strings.—The members receiving the ends of the steps to which they are usually housed and wedged, as shown in Figs. 741 to 742, are known as close strings. Strings adjacent to walls are known as wall strings, the remainder as outer strings. If, instead of being housed and wedged to the outer strings, steps are fitted, as shown in Figs. 735 to 736, they are known as cut and mitred. The latter have thin shaped brackets planted on outer strings, and one edge mitred to riser for effect, as shown in Figs. 735 to 736.

Pitch of Stairs.—It is found in practice that the best pitch for stairs is that inclination which by twicing the rise and adding the going equals 23. This agrees very well with the French theory: The labour of moving vertically is about twice that of moving horizontally, if the average human stride be taken as 23 inches.

Rise.	Tread.
$5\frac{1}{2}$	12
6	11
$6\frac{1}{2}$	10
7	9
$7\frac{1}{2}$	8

The limits of the variations of steps are from $9'' \times 7''$ to $11'' \times 6''$ for ordinary purposes, and are the most useful in practice, the former measurement for ordinary and the latter for more important buildings.

Width of Stairs.—The minimum width of stairs should admit of two persons conveniently passing each other. This

cannot conveniently be done under 2 ft. 9 in., but 3 feet is the more common dimension of all ordinary work of a good character.

The winders should have at least the width of the fliers on a curved line 15 inches from the centre of the newel.

Height of Straight Flights.—To prevent giddiness the number of steps will vary with the pitch of the stair, 8 feet being the maximum vertical rise without a landing or turn. Greater heights should only be reached by flat pitches, such as 11″ × 6″ or 12″ × 5½″.

Wood for Stairs.—In ordinary practice northern pine is used for all parts of stairs, but oak and teak are used for more important work, and are better for resisting great wear.

Pitch-pine is hardly suitable, although often used for ornamental effect for all parts of stairs, but unless thoroughly seasoned it shrinks and the joints open.

Italian walnut, for its colour and figure, is sometimes utilised for strings, balusters, newels, and handrails of stairs. Mahogany and teak are preferable for handrails, as they are durable, ornamental, and take a good polish.

Framing and Setting-out of Stairs.—The pitch and going are first determined, and a pitch-board made as shown in Fig. 744. The strings and newels are then dressed and set out by the aid of the pitch-board and plan of stairs; these are then housed, and the strings tenoned to the newels.

The risers and treads are prepared, fitted and glued; blocked to each other they form steps, and when set are fitted into the strings. After all have been fitted together the oblique tenons of the strings are glued and dowelled to the short newel, as shown in Fig. 721 (the longer newels not being secured until the work is in position, owing to the difficulty of transport); the steps are wedged to the strings, which then are ready to be fixed, and when *in situ* the rough carriages are fixed beneath, as shown in Figs. 738 to 742; to prevent distortion of the steps when subjected to rough usage, pieces of wood termed brackets,

Fig. 721.

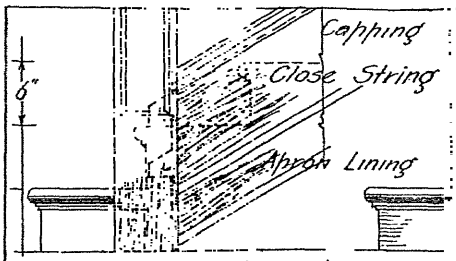


Fig. 725.

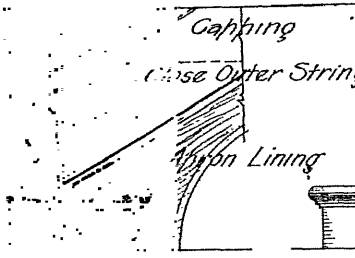
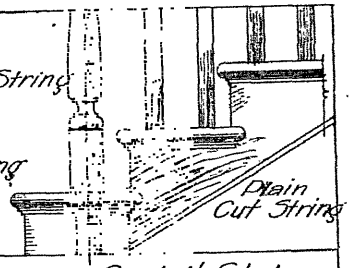
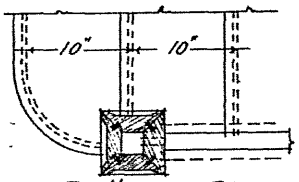


Fig. 731.



Side Elevations of Bullnose Rounded & Curtail Steps

Fig. 722.



Bullnose Step

Fig. 724.

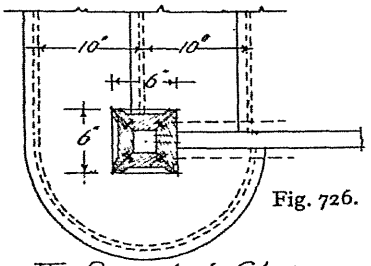
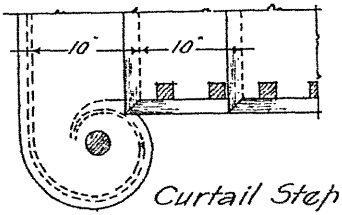


Fig. 726.

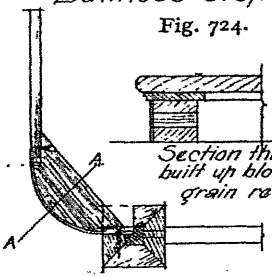
Rounded Step

Fig. 732.



Curtail Step

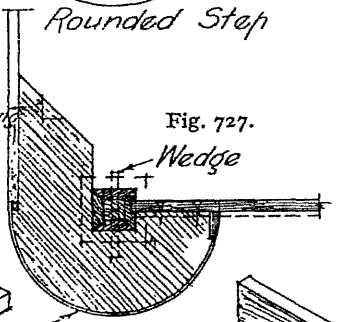
Fig. 723.



Section through built up block showing grain reversed

Block for Bullnose

Fig. 727.



Block for Rounded

Fig. 733.

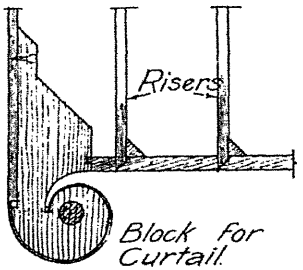
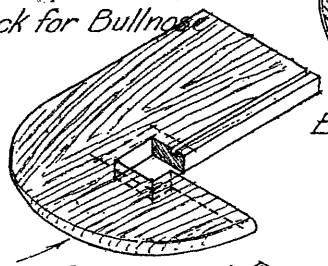
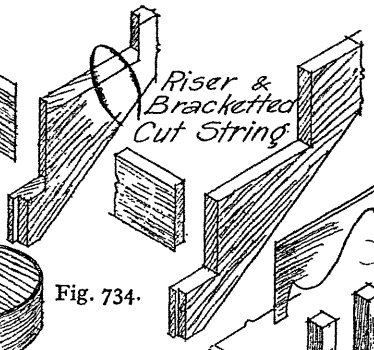


Fig. 728.



Tread for Rounded Step

Fig. 735.



Riser & Plain Cut String

Fig. 734.

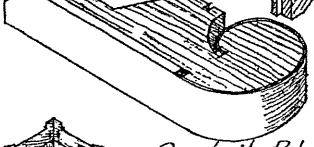
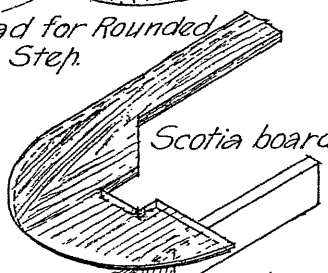


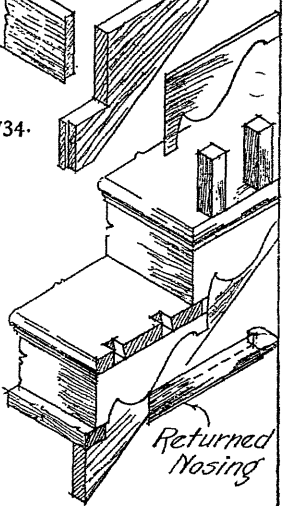
Fig. 729.



Curtail Block

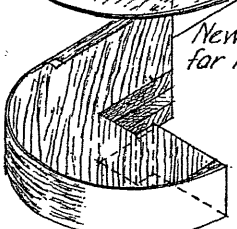
Built up Newel

Fig. 736.



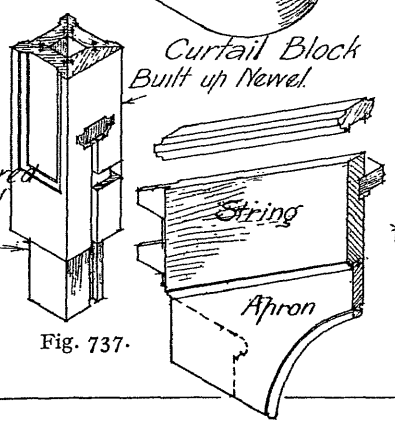
Newel prepared for Rounded Step

Fig. 730.



Block for Rounded Step

Fig. 737.



Figs. 721—737.

[Between pages 876 and 877

usually 1 inch in thickness, are cut and fitted to the underside of the step and spiked to the rough carriages and cut flush with the underside of the latter, as shown in Fig. 741.

Classification.—Stairs are of two kinds :—(*a*) Newel, (*b*) well.

Under (*a*) come the straight, dog-legged, solid newel and circular or rectangular.

Under (*b*) are included the open newel and the geometrical.

Straight stairs are useful for long narrow chambers ; dog-legged for staircases the width of which is slightly more than that of two stairs ; geometrical or open newel for those chambers of a greater width.

Open newel stairs present the best appearance and are strong.

Geometrical stairs require care and a good deal of skill in their construction ; they are not so imposing as the open newel, and are comparatively weak ; they were extensively used in middle-class houses of the Georgian period.

Newel Stairs.—Figs. 738 and 743 show plan section, elevation, and details of steps adaptable to newel stairs, showing treads, risers, rough brackets, rough carriages, wall and outer strings, cappings, apron lining, and plaster soffit.

Dog-legged Stairs.—Fig. 717 shows the outline plan of typical dog-legged stairs, with bull-nosed step, fliers, quarter space landing and three winders.

Figs. 738 to 743 give the working drawings showing pitch-board, story-rod, handrail with ramp and swansneck, and showing accurately all the necessary details.

Newels.—The posts into which the outer strings are framed at each turn in the stair are termed newels. These are usually square at the parts where the strings and rails are framed into them. They may be turned at the intermediate parts. They are made from stuff 4 inches square and upwards. Square newels more than 5 inches square are usually built up. When panelled they should be made from material at least 2 inches in thickness,

mitred, tongued and grooved at the angles. Hollow newels stand much better than solid, the latter having a tendency to develop shakes. Figs. 721 to 726 show the newel tenoned into the lower step, and Figs. 725 and 737 show the joint, between the lower end of the string and the newel. Figs. 731 to 733 show the fixing of the lower newel into the curtail step, and Figs. 735 to 736 details of a cut string.

Fig. 718 is an outline plan of an open newel stair, and Figs. 744 to 748 give the working drawings and details of an open newel stair with two quarter-space landings. Where it is possible to continue the newel to the floor as in the lower landing it is only necessary to arrange the supports as ordinary joists. Where it is not possible to continue the newel to the floor, the joists are framed and cantilevered as shown in Fig. 745. Fig. 745 is the plan, and shows the arrangement of the carriage pieces and trimmers. Fig. 746 shows an enlarged section through the fliers, Fig. 747 a section through the wall string, and Fig. 748 a section through a framed and panelled outer string with capping and rail.

Geometrical Stairs.—Fig. 719 shows a plan of a type of geometrical stairs with curtail steps, wreaths about wells, quarter-space landing and winders.

Figs. 749 to 754 give all the necessary drawings for such a staircase.

The string wreaths are constructed by making a centre upon which the portion of string to form the wreath, which has been already set out and sunk, is bent and temporarily fixed, face to the centre, to which upright staves with radiating joints are fitted and rubbed with glue, and on the unseen internal face of which canvas is glued to increase the rigidity and tenacity.

This when set is released from the centre and is cut to the set out marks and tongued to the grooved string as shown in Fig. 749. The straight as well as the curved portions of the string are then secured together by pieces of stuff arranged on the gib and cotter principle, similar to that shown in Fig. 930 in the *Elementary Course*, and known as the Counter Cramp.

Figs. 752 and 754 give the plan and elevation, showing

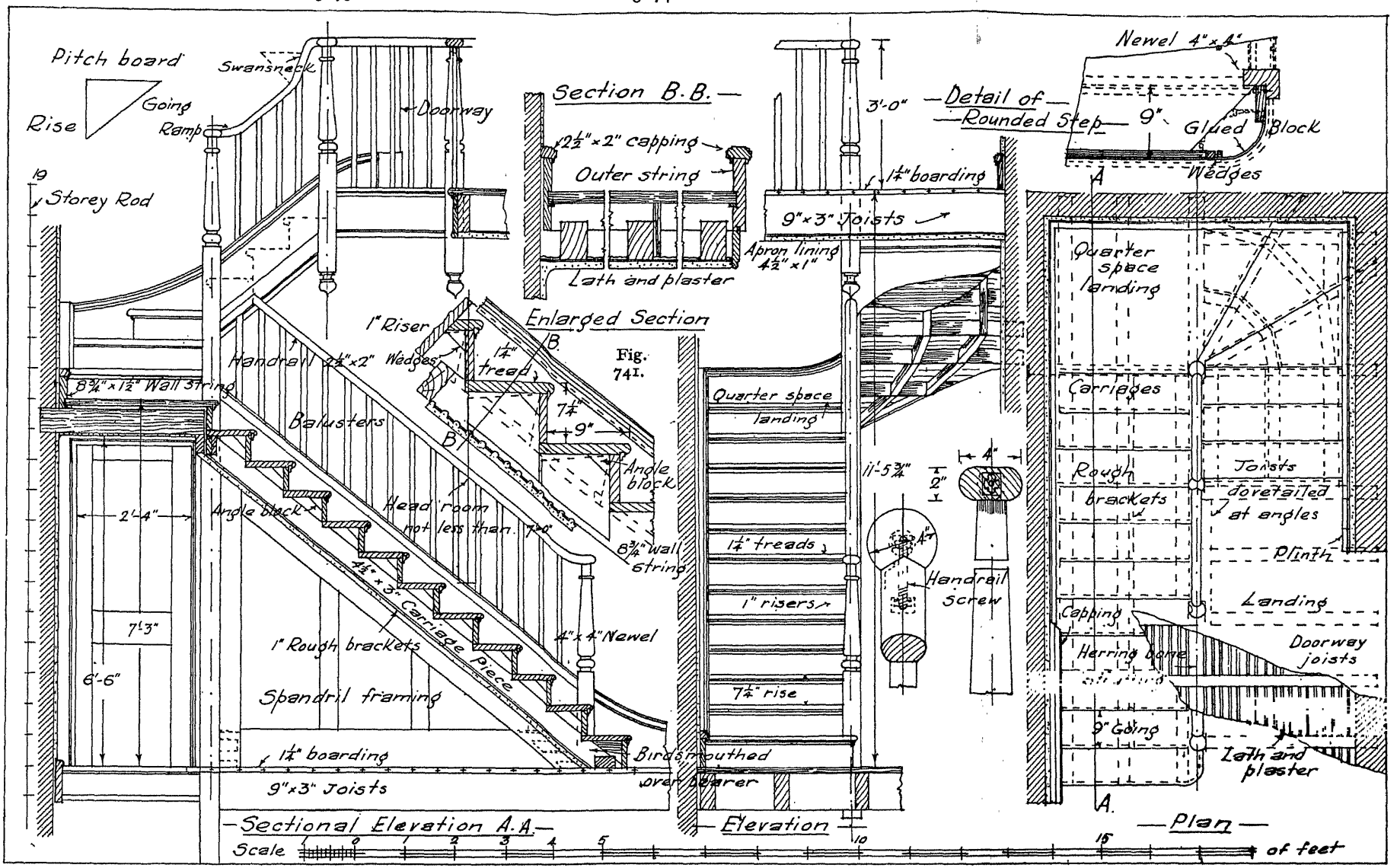


Fig. 738. Fig. 742.

Fig. 740. Fig. 743. Fig. 739.

Figs. 738—743.

[Between pages 878 and 879.

the junction of risers, treads, string and straight balusters of the outer cut string. The lower ends of the iron balusters are sometimes forged and screwed to the string, which has a block to strengthen it and receive the screws at that part. The thin ornamental bracket is planted on the string mitred with the riser and covering the baluster end. Bracket balusters similar to those shown in Fig. 752 are often fixed to the face of the string.

Handrailing.—The contour of the handrails in dog-legged stairs follows the line of the string, being ramped, mitred and fixed to newel caps by double-nutted screws, as shown in Fig. 743, or tenoned to newels, as shown in Fig. 744.

The handrails for open newel stairs are usually straight, and are tenoned and dowelled to newels, the heads of the latter usually being turned. Where handrails are used on the wall side they may be ramped.

In geometrical stairs the handrails should be constructed to present a graceful appearance, which effect is best obtained at a minimum cost by setting out the handrails and stairs on the tangent system.

Tangent-helical-joint at Springing Point System.—The production of handrails for continuous strings is accomplished by conceiving the centre line of rails enclosed by a series of lines tangent to the curve, which should, wherever the plan is circular, be a helix if possible, and by placing the joints at the points of contact made by the tangent lines and the curves. The following drawings are necessary :

(1) The plan with centre line of handrail and tangents, as shown in Figs. 755 and 773.

(2) The developments of the tangents, as shown in Figs. 761 to 770.

(3) The development of the face of the string on a vertical plane, as shown in Figs. 762 and 771.

(4) The face mould, for preparing the cylindrical surfaces of the rail, as shown in Figs. 763 and 772.

(5) The bevels, as shown in Figs. 764 and 766.

(6) The development of the falling moulds, or the cylindrical surfaces of the rail in all cases where the helical curve is departed from.

Setting out Handrails.—In setting out the work the following principles should be rigidy adhered to; and all rule of thumb methods discarded; all lines and processes should admit of geometrical proof, thus avoiding ambiguity and producing the best workmanship and scientific results in the most economical manner :—

First.—Any pair of tangents on any certain cylinder can have only one falling line. In geometrical language the falling line or curve is the intersection of the cylinder with the plane containing the tangents.

Second.—If two tangents about a circle in plan when developed form a straight line, the falling line will be a helical curve, and when developed will be a straight line.

Third.—It is desirable to have the stairs about the well-holes symmetrical; this can be done in the case of the half-space landing, or where winders occur all round the well or in a quarter-turn.

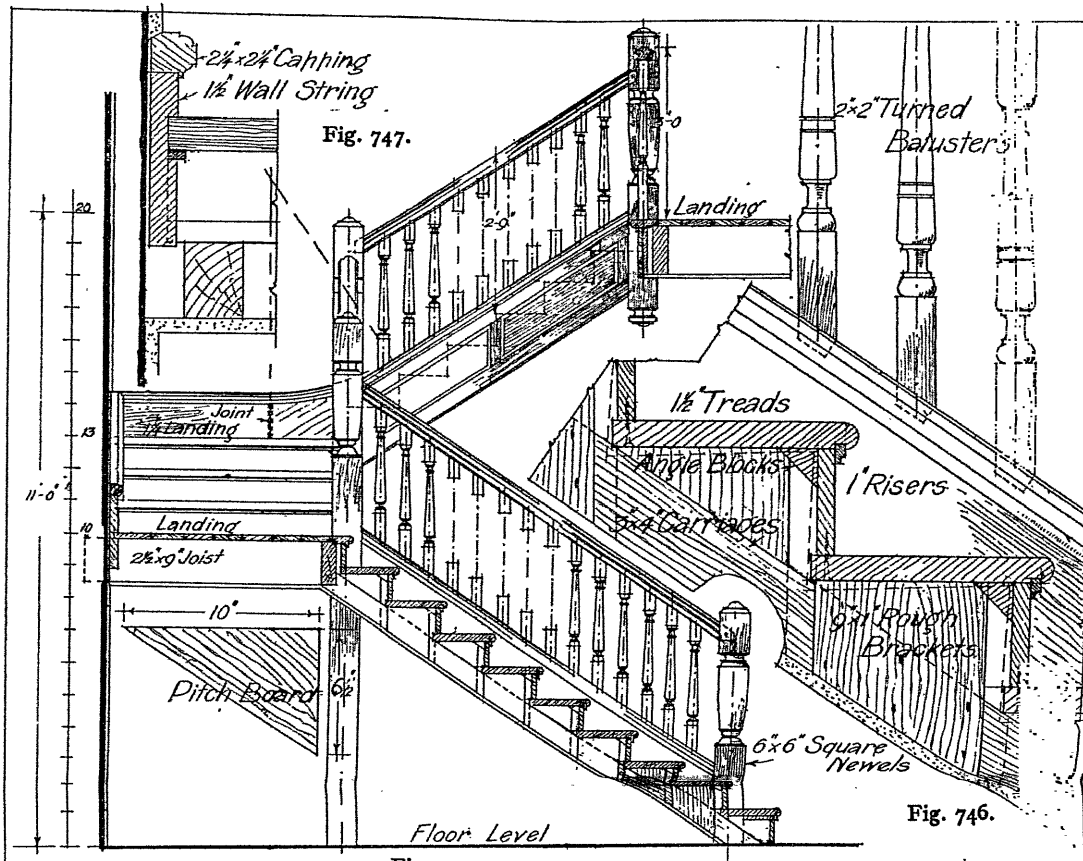
Fourth.—In the case of quarter-space landing and winders the developed falling line for the portion over the winders can be made straight; for the other half the rail will be of double curvature.

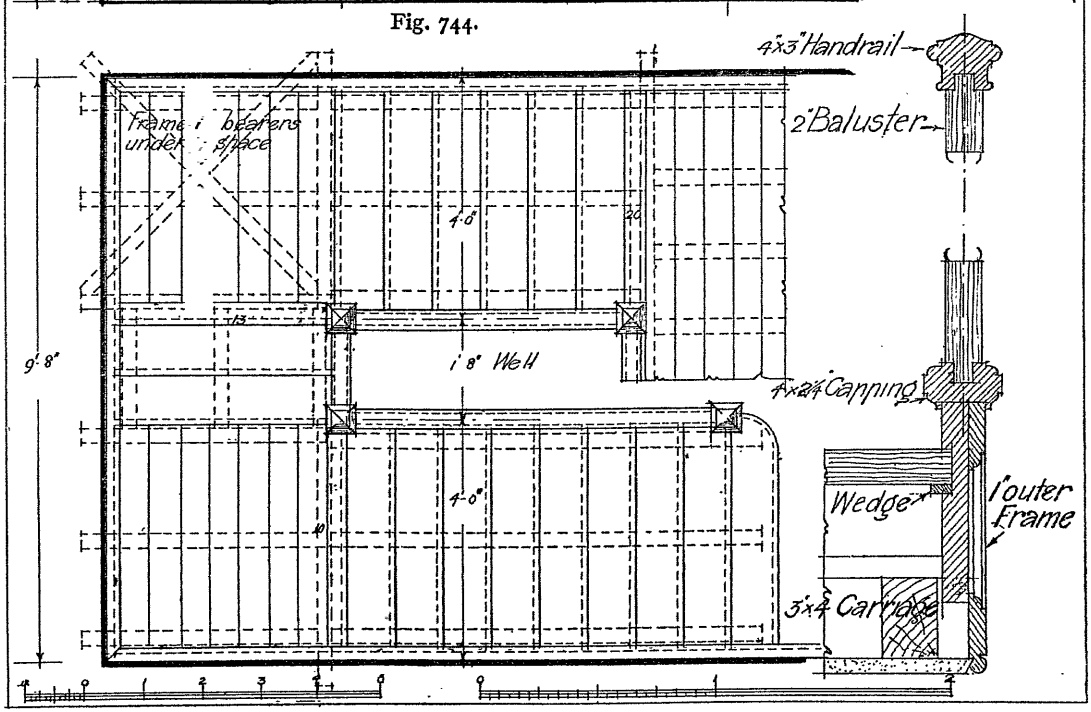
Fifth.—The plane of the joints should in every case be at right angles to the tangent lines; therefore joints should always be arranged at a point of contact of the curved line with the tangent; in other words, the springing point as indicated in plans and elevations, Figs. 755, 756 and 765.

Sixth.—All easements between straight and curved portions of the rail to be made on the rail, the centre line of which is straight in plan to avoid double curvature at any part.

There are four general types which cover all the ordinary cases met with in practice :—

First.—The scroll.
Second.—The wreath, about a half-space landing.
Third.—The wreath, about a half space with six winders.
Fourth.—The wreath, about a half turn with quarter-space landing and three winders.





Figs. 744—748. [*Between pages* 880 *and* 881.

Fig. 753. Fig. 754.

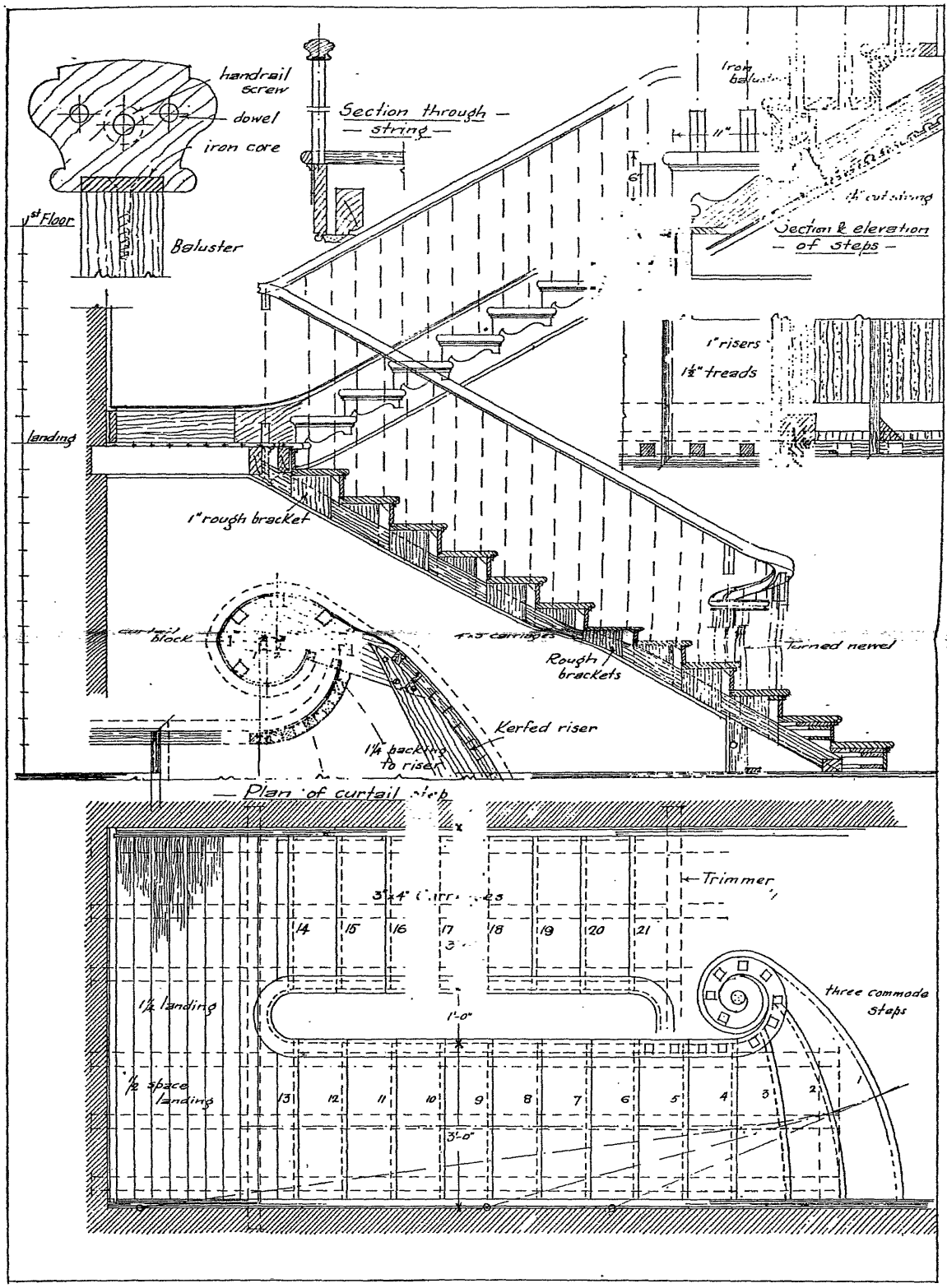


Fig. 752.

Fig. 751.

Fig. 749.

Fig. 750.

Figs. 749—754.

[*Between pages* 880 *and* 881.

Fig. 756. Fig. 757.

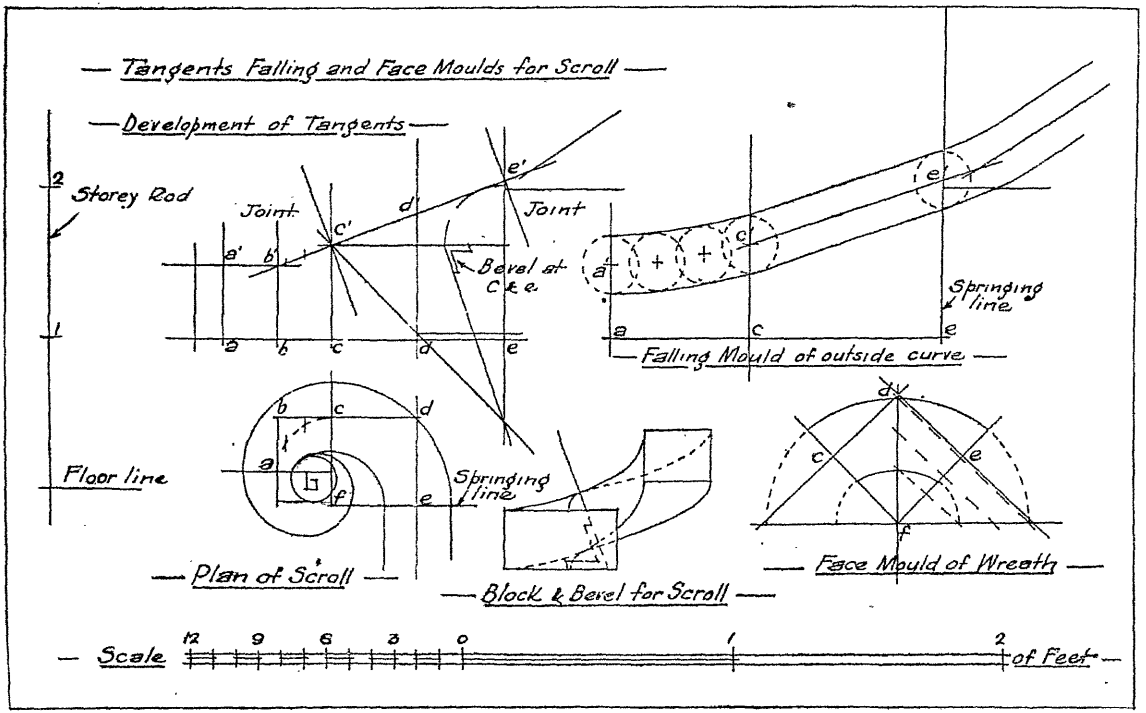


Fig. 755. Fig. 759. Fig. 758.

Figs. 755—759.

1. *The Scroll.*—The method of procedure is as follows: First draw the plan of the scroll and fix the position of the tangent lines, develop the tangents and the plane containing the face of the string. In all cases the steps should be arranged with regularity and symmetry. Draw the development of the rail above the string plane, taking care in any wreath portion to arrange the developed rail as straight, in order that when it is worked to its cylindrical form it will be a true helix. It is usual to make the scroll or block half a riser higher than the ordinary part of the rail; this causes a variation in the pitch of the rail, necessitating easements at the points *c* and *e* in Figs. 755 and 757. Having fixed on the development of the rail the heights of the points *c* and *d*, project them across on to the development of the tangents. On the latter drawing, set out the tangent *e'd'c'* and produce to *b'*, at *b'* draw the tangent *b'a'* horizontal. At a point about 3 inches from *e'* set up the centre line of the rail at the pitch of the stairs.

Prepare the face mould; from the plan draw the Fig. 758, *F, c, d, e.* The true length of the line *Fd* is shown on plan; the true lengths of the sides of the quadrilateral *fc, fe, de,* and *dc* can be obtained from the elevation of the tangents; the quadrilateral containing the springing and level lines of the face mould can then be drawn as shown in Fig. 758. The face mould is in all cases a portion of a section of a hollow cylinder, and, therefore, a portion of an ellipse, as the plane containing the tangents is inclined. As the two tangents *cd* and *de* are equally inclined the face mould will be symmetrically disposed about the minor axis *fd.* The lengths of the minor axes, or level lines, can be obtained from the plan (Fig. 755). The major axes, the inclinations of which are always the pitch of the planes, are obtained by drawing lines from the extremities of the minor axes parallel to the tangent lines *de* and *dc*; having the major and minor axes the ellipses may then be drawn, and the face mould will be that portion contained between the lines *fc* and *fe* (Fig. 758). For the preparation of the scroll block, let an elevation be drawn parallel to the tangent *bd* as shown in Fig. 759. The bevel for the joint can be obtained from the elevation of the tangents (Fig. 756). It must be particularly noted that all joints

Fig. 761.

Fig. 762.

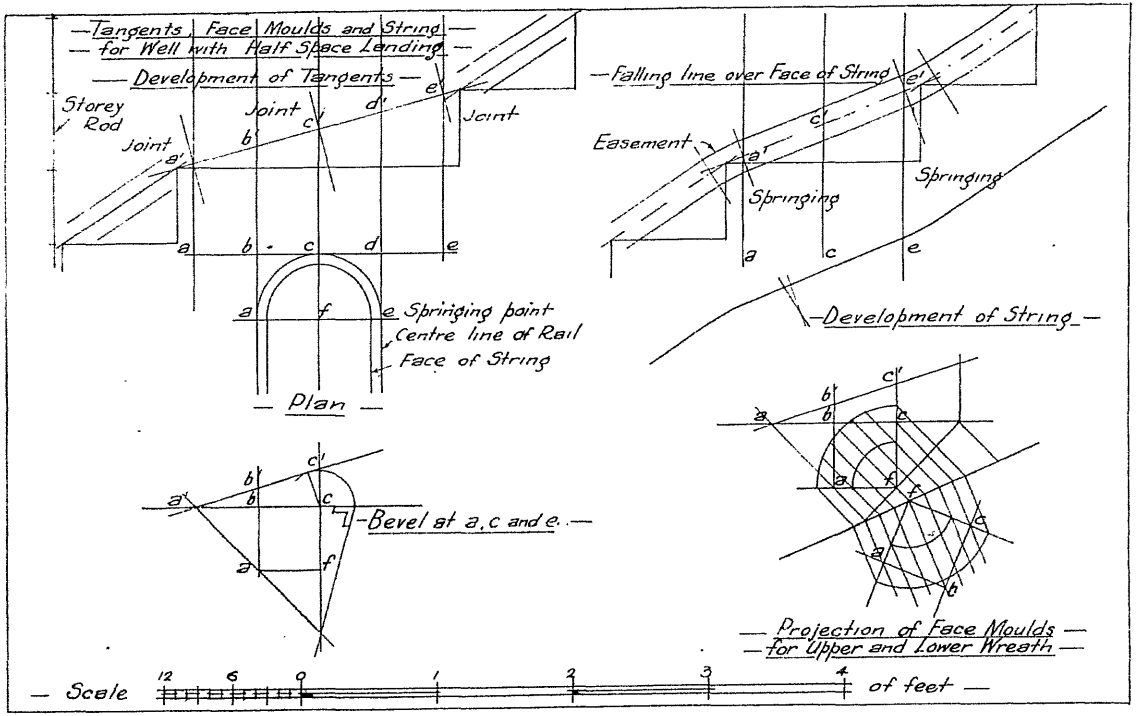


Fig. 760.

Fig. 764.

Fig. 763.

Figs. 760—764.

in this system are at right angles to their respective tangents, and are made before the stuff is shaped.

Bevels for Wreaths.—The bevels to be used with the face moulds represent the dihedral angle at each extremity between the vertical plane containing the tangent and the plane containing the centre line of the rail.

2. *Wreath about Half-space Landing.*—Draw the plan, as shown in Fig. 760, showing centre line of the wreath and face of string. Draw tangent lines *a*, *b*, *c*, *d*, *e*, enclosing the centre line of rail; set out the elevation of the tangents developed on a plane. Let the landing extend into the straight portion of the stair about 3 inches beyond each springing line, draw the fliers and the centre line of the rail touching the nosings, then set out the tangent from the lower to the upper nosing as shown on Fig. 761. As the tangents form a straight line the resulting wreath will be a helix; from these tangent lines draw the face moulds as before described and shown in Fig. 763. As the tangents are symmetrical about the centre line *c*, one face mould will be true for the upper and lower portions of the wreath. Develop, as before described, the face of string, as shown in Fig. 762. The section of the rail immediately above the string is also shown, and gives the method of obtaining the easements between the wreath and the straight portions of the rail. The methods of obtaining the bevels are shown in Fig. 764 to prevent ambiguity.

3. *Wreath with Six Winders.*—The procedure in this example is identical with case No. 2, the winders being set out symmetrically about the centre line and projecting about 3 inches into the straight portion of the rail at the upper and lower springing points. Figs. 765 to 768 show a plan, development of tangents, development of face of string, face mould and bevels for this example.

4. *Wreath about a Half Turn, with Quarter-space Landing and Three Winders.*—Draw the plan showing the face of the string; centre line inside and outside of rail, with the enclosing tangents *a*, *b*, *c*, *d*, and *e*, as shown in Fig. 769.

Fig. 766.

Fig. 767.

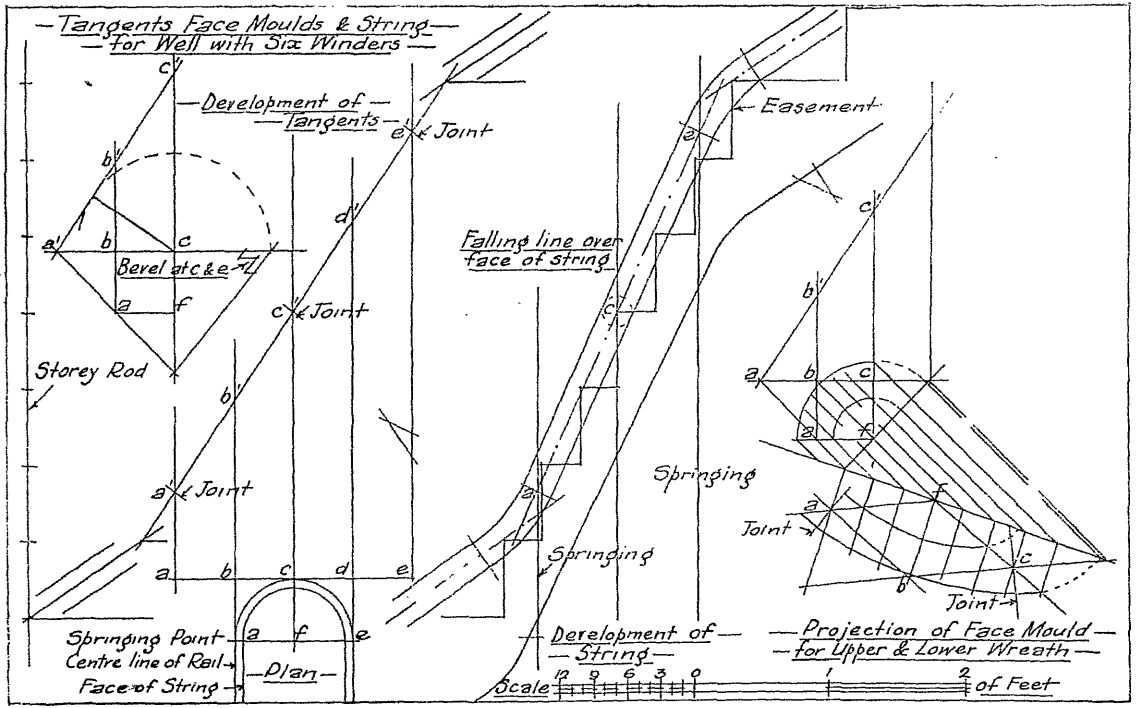


Fig. 765.

Fig. 768.

Figs. 765—768.

Fig. 770. Fig. 771.

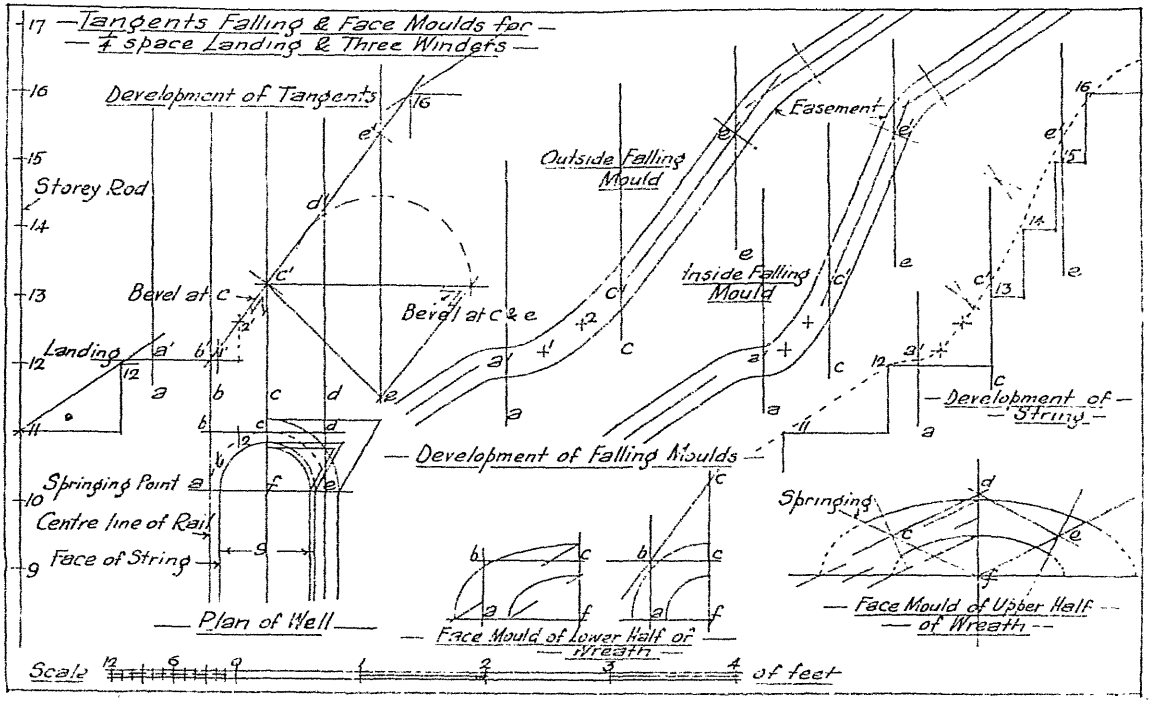


Fig. 769. Fig. 773. Fig. 772.

Figs. 769—773.

Draw the elevation of the tangents with their developments; set out the lower steps marked 11 and 12, keeping the nosing of 12 about 3 inches beyond the springing point and step 16, keeping the nosing about 3 inches beyond tangent line *e*. Draw the elevation of the tangent *ab* horizontally, and keeping it about 1 inch above the level of the nosing No. 12. Draw the tangents *b'c'*, *d'e'* commencing from *b'* and joining the centre line of the upper portion of the rail at the nosing 16, as shown in Fig. 770. By this arrangement the portion of the wreath over the

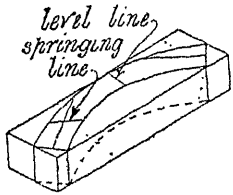


Fig. 774.

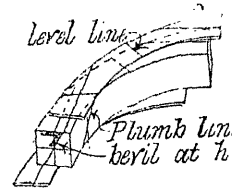


Fig. 775.

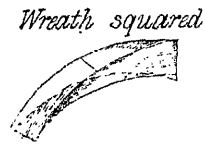


Fig. 776.

winders will form a helical curve; the face mould for this may be prepared and the rail produced without the aid of a falling mould. The lower portion of the wreath will have a double curve, and will require a face mould to work its cylindrical faces and two falling moulds to give to it the vertical curves. The method of preparing the face mould will be as previously described for No. 1 case. The method of producing the falling moulds is as follows: Develop the inside and the outside cylinders; the heights of the points *c* and *e* can be projected from the developments of the tangents; as shown in Fig. 771 the upper portion of the wreath can be drawn in direct, as, being a helical curve, it will show as a straight line in its development. Complete the ease-

ment for the upper portion of the wreath on the straight portion of the rail in plan. Draw the lower straight portion of the rail with its easement, as shown in Fig. 771. The method of connecting these two parts of the rail will be as follows: Divide the centre line of the lower portion of the rail into three equal parts, *a*—1, 1—2, 2—*c*, draw the vertical lines passing through 1—2 in the vertical parts of the rail, then obtain the heights of the points 1 and 2. The points 1 and 2 are obtained in the plane bounded by the tangents *a*—*b*, and *b*—*c* as follows: Project the points 1 and 2 on the tangent *b'c'*; this will be the heights of the points 1 and 2, because the tangent *a'b'* is horizontal. Project these points on to the two developments of the rail, then having four points in the lower portion of the rail the curve may be drawn, as shown in Fig. 771. Figs. 772 and 773 show the face moulds.

The line marked level line in the Figs. 774 to 776 is that part at which the rail may be imagined to commence twisting in the opposite direction.

The method of setting out rail on the dressed plank is shown in Fig. 774; and of sliding the duplicated moulds to get the twist is shown in Fig. 775; and the view of the finished portion of wreath is shown in Fig. 776.

CHAPTER XXV

SANITATION

Definition.—Sanitation is defined as the devising and application of means for the promotion or preservation of health. The principles or rules upon which sanitation is based form the science of hygiene or sanitary science.

In its broad aspects, sanitation is concerned with food, water, air, clothing, living quarters, work places and their environment, and the general amenities of the district in which the community exists, including the provision of open spaces and facilities for recreation. Facilities must also be available in order that the cleanliness of the person may be obtained.

In relation to buildings, sanitation requires consideration in the following, among other respects :—

(*a*) Adequate lighting must be provided to all parts of the building.

(*b*) Pure water must be available in sufficient quantity.

(*c*) Food must be safeguarded against deterioration.

(*d*) Good ventilation must exist in order that the air which has become polluted may be efficiently replaced.

(*e*) Offensive and waste matters must be speedily removed by efficient water closets, etc., or, in the case of refuse, must be so kept pending removal as not to become injurious to health or otherwise objectionable.

(*f*) For cleanliness of the person, sufficient bathing and washing appliances must be readily available.

Before, however, considering the conditions of the building, it is necessary to consider the circumstances of the site. Not only must the characteristics of the soil be ascertained, as will already have been done (see Chapter XI.) in regard to its structural suitability, investigation must also be made as to its hygienic suitability.

The various soils already referred to have each their special characteristics in regard to health.

Rock will usually provide a healthy site. Chalk will be found generally to be healthy, but if high lying, is unfavourable in times of drought owing to the deterioration of surrounding vegetation, and if low lying may be water-logged in wet seasons. Gravel is usually considered good, but as it is found either as valley gravel or contained in depressions in an underlying stratum, it may in either case be subject to great variation of level of the *ground water*. Sand has similar characteristics. Clay has long been regarded as unfavourable to health, but recent experience causes us to qualify that opinion. Healthiness of any site will depend in large measure upon the drainage, and if clay has a gentle slope by which the surface is kept dry, it may prove a more healthy site than one of gravel in which drainage conditions are unfavourable.

Ground water, above referred to, is the water contained in the interstices of the soil. The level up to which the ground is so saturated should be kept low and constant by means of subsoil drainage, described in the chapter on Foundations. Fluctuation of the ground water level causes the alternate entry and discharge of *ground air*, which militates against hygienic conditions.

Lighting.—The first condition for hygienic buildings is usually satisfied by complying with the provisions of the Model Bye-Laws, which are as follows :—

"88.—(1) A sufficient number of windows shall be constructed in the wall of every storey of a domestic building in such a manner and in such a position that each of the windows affords effectual means of ventilation by direct communication with the external air.

(2) Every habitable room shall be provided with a window or windows which shall open directly into the external air and—

"(*a*) have a total area not less than *one-tenth* of the floor area of the room ; and

"(*b*) be so constructed that a total area not less than *one-twentieth* of the floor area of the room may be opened, and so that at least to the extent of this requirement the windows can be opened at the top."

Similar provisions are made in the London County Council Bye-Laws, para. 149, London Building Acts, 1930-1939.

Provision should also be made for staircases and corridors to be efficiently lighted. A convenient position for lighting the former is from the top ; but all landings should have windows wherever possible. Corridors should be lighted at the ends, and, if very long and it is convenient to do so, by ceiling or skylights. All windows should be capable of being opened for air currents to pass through when desired.

Heights of and open spaces about buildings are dealt with in L.B.A., 1930, ss. 43, 44, 45, 46, 47, 50, in conjunction with L.B.A., 1939, ss. 4, 140.

Water Supply.—The sources of water supply have been classified as follows : (1) upland waters, (2) rivers, (3) springs, (4) wells.

In country districts it may be necessary to provide a water supply from natural sources, in which case recourse will probably be made to a well. Wells, again, are classified : (1) deep, (2) shallow, (3) artesian. The distinction between (1) and (2) is not one of depth by measurement but whether the water supply is obtained from a surface stratum or from a stratum between which and the surface an impervious layer is interposed. The former is a shallow well whatever the dimension from surface to water level may be.

By reason of the absence of the impervious layer a shallow well is exposed to contamination from surface conditions and should not be favourably regarded, except after exhaustive investigation, as a source of dietetic water.

Water distribution, however, is now so general in urban, and in many rural districts that wells may in most cases be avoided.

Quantity of Water Required.—This will depend very largely upon the system of sewage disposal adopted. With a water-carried system, the frequent flushing of the water closet apparatus each day by each person will make a corresponding increase in the water requirements ; with water-carried systems baths are more generally installed, and as a bath requires from 12 to 30 gallons of water a further increased supply of water becomes necessary.

In rural communities it has been stated that the consumption of water does not exceed 2 gallons per head per

day, and that 10 gallons is an ample provision. In urban districts having water-carried sewage systems, the figure of 30 gallons per head per day is usually quoted, although the consumption of large towns is found to exceed this, varying from 27 gallons (Birmingham) to 65 gallons (Glasgow). The consumption will be affected by the state of repair in which the mains and fittings are maintained.

In considering the consumption, it must be remembered that water is not all used for domestic purposes. The average figure of 30 gallons already quoted may be considered as divisible among the following heads :—

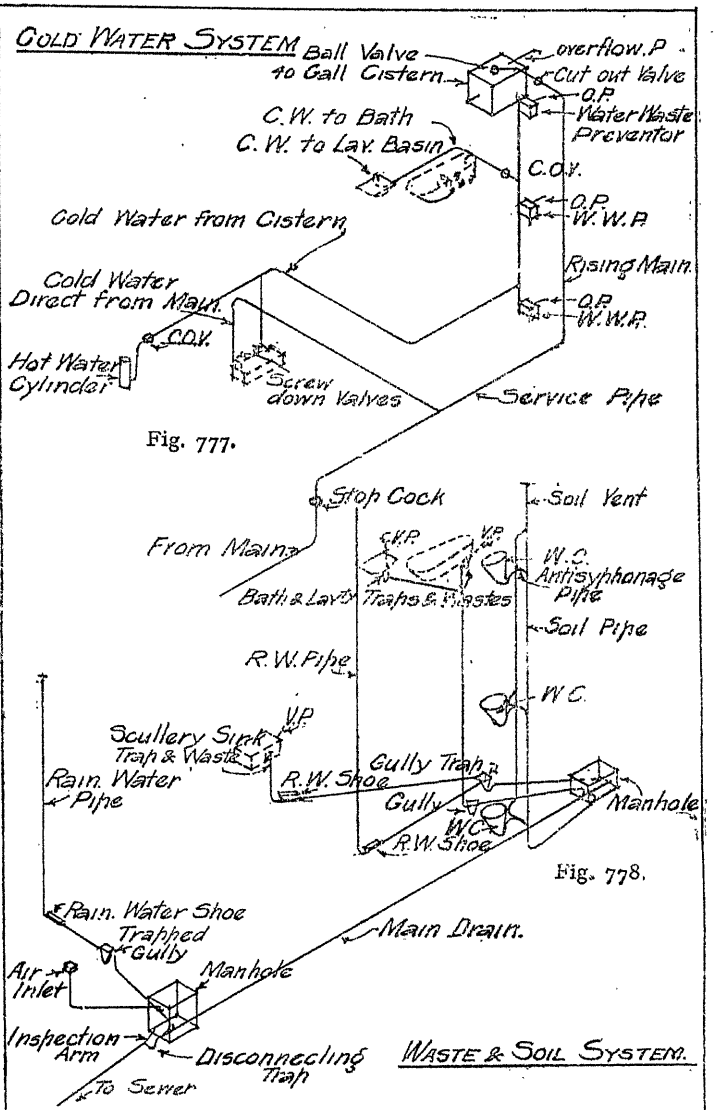
Domestic	say	10 to 12 gallons.
Non-domestic :		
Trade		12 gallons.
Municipal, street watering, fires, etc.		6 gallons.
		30 gallons.

Quality of Water.—The quality of waters in their natural state may differ very widely, and in advising upon a natural source of supply it would be necessary to make a careful examination of the source and the general conditions affecting it, as well as the water obtained ; it may also be necessary to obtain a chemical analysis. In the case of an engineered supply this will however rarely be necessary.

The condition of the water with regard to *hardness* should be known. A water is said to be *hard* when it is difficult to obtain a lather with soap. Comparative figures of hardness for different waters are obtained by finding the amount of standard soap solution required to produce a permanent lather. Waters of 5 degrees of hardness and under are considered soft, those over 12 degrees hard. A degree is 1 grain per gallon.

Temporary hardness is that which can be removed by boiling the water. It is caused by the presence in solution of calcium bicarbonate and magnesium bicarbonate. Carbon dioxide is driven off when the water is boiled, calcium carbonate and magnesium carbonate are precipitated and the water is softened. It is this precipitation that causes the *fur* in kettles and in the flow pipes from boilers.

Permanent hardness remains after boiling and is due to calcium and magnesium sulphates and/or chlorides.


COLD WATER SYSTEM
Ball Valve to Gall Cistern
overflow.P
Cut out Valve
O.P
Water Waste Preventor
C.W. to Bath
C.W. to Lav. Basin
C.O.V.
O.P
W.W.P
Cold Water from Cistern
Rising Main
Cold Water Direct from Main
O.P
W.W.P
C.O.V.
Hot Water Cylinder
Screw down Valves
Service Pipe
Fig. 777.
Stop Cock
Soil Vent
From Main
C.W.P
V.P
W.C. Antisyphonage Pipe
Bath & Lavs
Traps & Wastes
Soil Pipe
R.W. Pipe
W.C.
V.P
Scullery Sink Trap & Waste
Gully Trap
Rain Water Pipe
R.W. Shoe
Gully
Manhole
W.C.
R.W. Shoe
Fig. 778.
Rain Water Shoe
Trapped Gully
Main Drain.
Air Inlet
Manhole
Inspection Arm
Disconnecting Trap
WASTE & SOIL SYSTEM.
To Sewer

Figs. 777—778.

Two methods of softening in use are the *lime and soda* method and the *zeolite* process :—

(*a*) A properly adjusted mixture of lime and soda solution is added to the hard water and in the chemical changes that take place the lime removes the temporary and the soda the permanent hardness.

(*b*) Zeolites (sodium alumino-silicates) are a group of minerals which have the property of giving up their sodium in exchange for the calcium and magnesium in the salts producing hardness. When hard water is passed over the porous granular zeolite in the special apparatus used for this purpose, the calcium and magnesium salts are replaced by those of sodium and the water is softened. When all the sodium in the zeolite is used up its softening power is fully restored by flushing with a brine solution from which the zeolite regains its sodium. The calcium chloride formed is then washed away.

It should be noted that water softened by the above methods still contains solids in solution. The only method of obtaining pure water free from solids is by distillation. This is, for instance, the only way in which sea-water can be made drinkable. Distilled water tastes *flat*.

Pure or very soft water corrodes iron and dissolves lead from lead pipes. In districts where very soft water occurs, the water should be artificially hardened to about 5 degrees and copper tubing should be used.

Water supply pipes and fittings for the storage and distribution of water for dietetic purposes should be so arranged and fixed that the water is not brought into contact with contaminating influences. In towns, the water supply for drinking and culinary purposes should be drawn from the rising main pipe, and not stored in cisterns where the water may absorb noxious gases, or develop organisms injurious to health.

Water used for the cleansing and flushing of sanitary arrangements should not be taken directly from the main, but from a cistern in which the supply is automatically regulated by means of a ball valve.

"Every pipe hereafter laid or fixed in the interior of any dwelling-house for the conveyance of, or in connection with, the water of the Board must, unless with the consent

of the Board, if in contact with the ground, be of lead, but may otherwise be of lead, copper, or wrought iron, at the option of the consumer." (Regulations made under Metropolis Water Act, 1871.)

The Metropolitan Water Board requires that lead pipes must be as follows :—

Internal diameter of pipe in inches.		Weight of pipe in lbs. per lineal yard.	
	⅝ inch		5 lbs.
	¾ ,,		6 ,,
	⅞ ,,		7½ ,,
	1 ,,		9 ,,
	1¼ ,,		12 ,,
	1½ ,,		16 ,,

Healthy Air.—Atmospheric air is vitiated by the following causes : (1) damp sites, walls, or roofs ; (2) decomposition of material used in construction, notably the timber ; (3) enclosed spaces which are neither airtight nor well ventilated, such as the spaces enclosed by hollow walls ; and by (4) the overcharging of the atmosphere with carbon dioxide caused by respiration, and the body exhalations of human beings or animal bodies, in an ill-ventilated chamber ; or (5) by saturating the atmosphere with noxious gases from decomposing organic waste matter contained in dustbins, privies, etc.

Construction.—In order to obviate defects arising from construction, hygienic methods must be adopted. The various details have already been dealt with in the respective chapters, but they are now summarized :—

Site.—To be covered with an impervious layer.
Walls.—To have efficient damp course, *e.g.* :—
Horizontal : Slate in cement, bituminous felt, lead, asphalt, vitrified stoneware.
Vertical : Slate in cement, asphalt, bituminous compounds in brickwork.
Dry Areas : Open, or enclosed and adequately ventilated.
Brickwork : Well-burnt, hard and sound, properly bonded.
Hollow walls.
Weather facings—tile hung, compo.
Stone : Waterproofed if necessary.
Chimneys.—Horizontal damp courses are necessary to prevent downward percolation.
Roof Materials.—Non-absorbent, non-conductors of heat.
Ventilation.—All timber ventilated against dry rot.

Ventilation.—This will be dealt with in a subsequent chapter (p. 983).

Disposal of House Refuse.—House refuse should not be allowed to accumulate and decompose in or near habitable dwellings; (1) in rural districts it should be speedily distributed over the land; (2) in towns, fixed dustbins should not be used, but in their place portable galvanized iron vessels with lids, and having a capacity of not more than 2 cubic feet, should be provided and placed in the open air in the rear of each building, and emptied daily; but they must not be cleared less frequently than once in every week, otherwise there will be great danger of the atmosphere being seriously contaminated by the decomposition of the contents. If emptied weekly it is usually found that it is necessary to have a content of $3\frac{1}{2}$ cubic feet in the portable vessel.

In rural districts there is little or no difficulty in disposing of the house refuse upon the land, but in London and large towns it is more sanitary to reduce it to ashes by burning in large furnaces or dust destructors; this method is, however, costly.

The great improvement in recent years in the design and construction of dust destructors in England renders it possible to reduce large quantities of refuse matter by incineration, and provided that the destructors are erected in non-residential areas, and away from public buildings, such as schools, churches, etc., they need not be any more objectionable than the output of an ordinary factory chimney, and up to the present this is the only satisfactory method from the hygienic standpoint for the disposal of house refuse, offal, etc. Developments are now being introduced by which mechanical separators are used to extract saleable articles, such as tins, bottles, bones, etc., leaving a small residue only for incineration.

Closets.—Classification.—Closets may be classified as follows:—

(1) Conservancy.
  (*a*) Cesspools.
  (*b*) Privies.
  (*c*) Earth closets.
(2) Water closets.

Conservancy Systems.—Cesspools.—Where a cesspool is used the closet was in antiquated forms set directly above

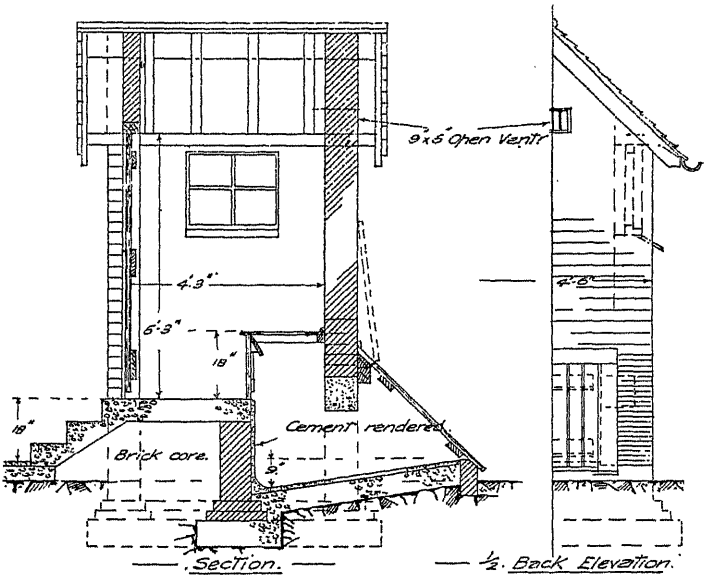


Fig. 779.

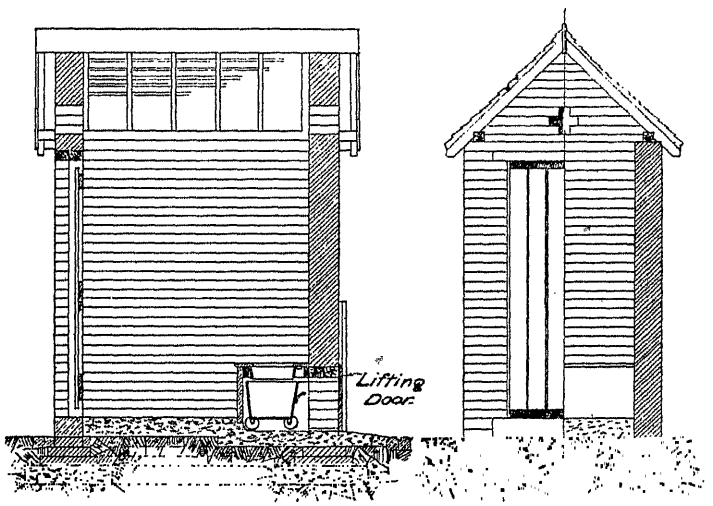


Fig. 780.

it. This form of construction will only in the rarest cases be permissible, and must be removed from the dwelling so that the cesspool is remote therefrom as required by the bye-laws. Cesspools may, however, be used in connection with a water-carried system, and should then be constructed as shown in Figs. 827 and 828.

Privies.—The privy is simply an enclosure, as shown in Fig. 779, with a pit beneath the seat, constructed in such a manner that it may be easily cleaned out. This is an unsanitary arrangement and should be avoided. To comply with the L.C.C. Drainage Byelaws, to which reference should be made, a privy must be 20 feet at least away from any dwellings or business structures and cleansed every week. But in towns, or anywhere in close proximity to buildings, a privy is a "nuisance" and dangerous to health, and therefore this form of closet should not be used.

Fig. 780 shows an arrangement with a movable receptacle, which can be emptied as frequently as it may be desired. A privy with a removable receptacle is the only type that should be employed.

Earth Closets.—An earth closet consists of an enclosure, built in a detached outbuilding, or abutting against the dead wall of a dwelling. The entrance should be outside, and there should not be any communication with the atmosphere inside the house. These closets should be provided either with an automatic arrangement, illustrated in Fig. 781, or with the means of adding by hand sufficient dried ashes, dried and powdered clay, dry earth, loam, or sawdust, to cover the deposit at each usage of the closet. Sand and gravel are of little value for deodorizing the deposit or absorbing liquid fæces.

Earth closets should only be provided in small country houses. The rooms in which they are fixed should be well ventilated, the walls and floors next to adjoining rooms made perfectly airtight and the entrances should be from passages or landings and not from any living-rooms.

The L.C.C. Drainage Byelaws require that two of the sides of an earth closet shall be external.

Earth closets are fitted with movable receptacles beneath the seats, and should be cleared out daily, or at least once in every three days. This is known as the pail system.

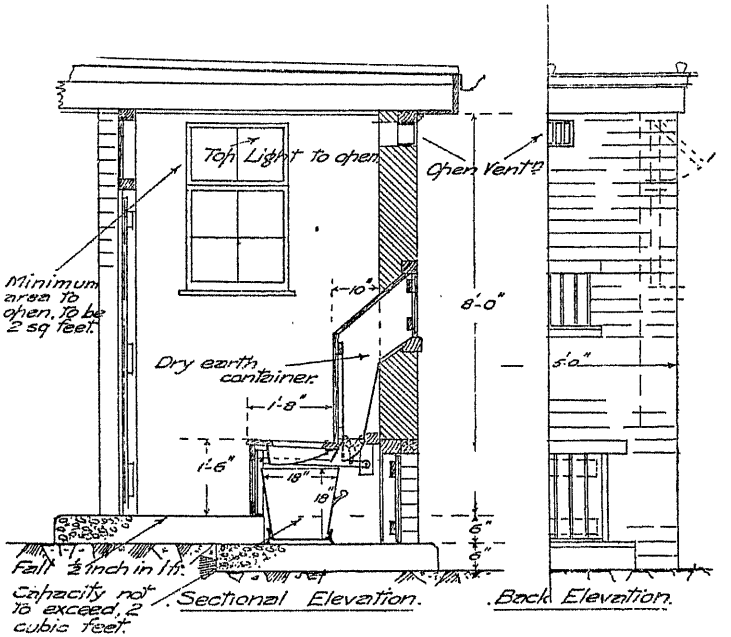


Fig. 781.

In towns the earth closet system is now superseded by the water-carriage system.

In villages and cottages the privies are sometimes converted to the earth-closet principle by using them as dust shoots or ash pits.

Water-Carriage System.—In a town where the water-carriage system for removing waste liquids is adopted complete systems of drains and sewers are necessary. The surface water and sewage matter are conveyed away by drains and sewers which may either be arranged on the "dual system" in which one series of sewers is provided for the surface water and another series for the sewage, or on the "combined" system where both surface water and sewage are carried by the same pipes.

The "absolutely separate" system has been advocated by some authorities, in which three series of pipes are sug-

gested, for the sewage, surface water and subsoil water respectively, and it is considered that under suitable conditions this would result in economy, as old culverts, etc., could be utilized for the surface water.

The dual system is often carried out in districts where the sewage is chemically or bacterially treated to save the expense of treating the surface water; but in the combined system advantage is taken of the surface water in times of heavy downpours to scour the drains and sewers, thus automatically ensuring that at least a periodical cleansing and water flushing of the drains is effected; in addition to this, two systems of drains in each dwelling-house is an additional expense. The combined system only is described here.

In order that the houses may be healthy it is usual to provide for—

(*a*) Constructing the means for conveying away all liquid waste in such a manner that every part liable to be fouled will be, by ordinary usage, and when flushed with water, as nearly as possible self-cleansing. The drains and pipes should also be periodically flushed with clean water, the object being to rapidly transport all offensive decomposing matter to the sewers.

(*b*) Complete disconnection of the house drains from the sewers or cesspools by the use of disconnecting traps.

(*c*) Disconnection of rain, bath waste, or other waste water pipes by causing them to discharge in the open air into properly constructed stoneware gully traps.

(*d*) Ventilating the drains, sewers, and foul water pipes by inducing currents of air to pass through and so flush every part of the interior of the drains and pipes. To obtain the maximum of efficiency there should be as few dead-ends as possible, and where these cannot be dispensed with, the drains and pipes should be made as short as possible.

All drains should be periodically inspected and cleansed. Where laid underneath a building the drains should be of earthenware enclosed in 6 inches of cement concrete, or of iron to prevent fracture either by settlement or crushing of the earth or surroundings. They should be laid in a straight line. Drains laid outside of buildings require to be bedded in concrete to the extent of half the diameter of the pipes (see Bye-laws).

Materials for Drains.—All drain pipes, channels, bends, and other appliances should be made as small as possible to be efficient, permitting of the maximum sphere having a clear way throughout its entire length, and the appliances should be of materials which are non-absorbent, smooth, durable and proof against the escape of drain air; the necessary joints and connections must be easily made, water-tight, proof against vibrations caused by mechanically driven vehicles, and durable.

Glazed stoneware and cast iron treated with a preservative solution are the two materials that most economically satisfy these requirements. The lengths of glazed stoneware are usually 2 feet, therefore necessitating a joint in every 2 feet of its length; gasket and neat Portland cement are the jointing materials.

Cast-iron pipes are made in lengths of 9 feet, thereby in long lengths of drains reducing the number of joints; gasket and metallic lead are the jointing materials.

Glazed stoneware pipes with Portland cement jointing may after a somewhat limited period become unsatisfactory owing to the contraction and expansion of the cement jointing, together with the damage resulting from earth movements. This applies particularly in the case of clay soils. Cast-iron pipes properly treated with metallic lead jointing are much more reliable, and the resistance to earth movements caused by natural conditions or by mechanically driven vehicles is much more satisfactory, and, therefore, in towns and under buildings are now often used, even though the initial cost may be considerably greater.

For forms, weights, properties, dimensions and tests for stoneware drain pipes, see B.S.S. 65. For stoneware drain fittings, traps, etc., see B.S.S. 539, and for cast-iron drain pipes see B.S.S. 437.

House Drains.—In a terrace house the drain should be laid straight under the house, so that it can be easily cleansed when required. In all detached and semi-detached houses the drains should be laid outside the house. All drains should be laid to such falls and gradients,

Joint between Stoneware and Stoneware
Stoneware
Portland Cement
Stoneware
Fig. 782.
Joint between Lead and Iron
Lead
Solder
Brass Thimble
Lead
Gasket
Iron
Stoneware
Portland Cement
Stoneware
Fig. 783.
Joint between Iron and Stoneware
Iron
Portland Cement
Gasket
Stoneware
Fig. 784.
Joint between Lead and Stoneware
Lead
Solder
Brass Thimble
Portland Cement
Gasket
Stoneware
Fig. 785.
Joint between Iron and Iron.
Iron
Lead Caulked
Gasket
Iron
Fig. 786.

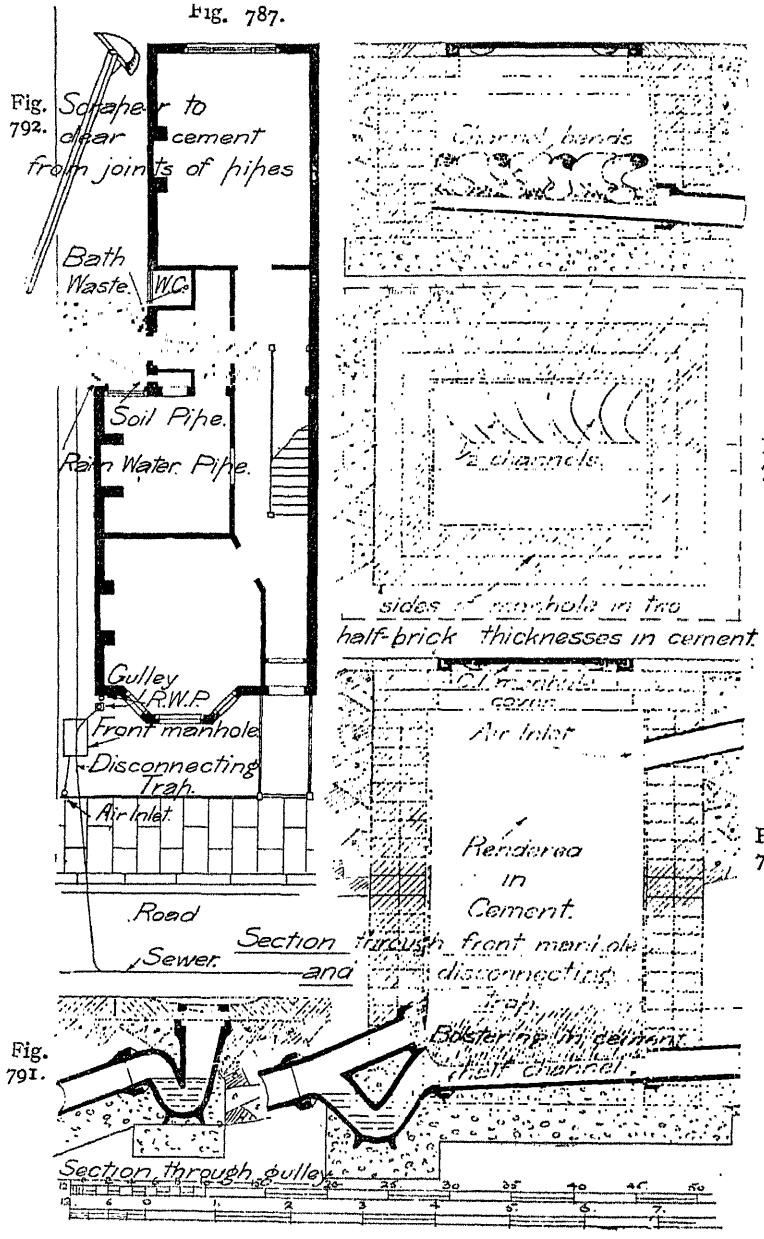
Figs. 782—786.

and be of such sizes, as to be as nearly as possible self-cleansing.

All pipes above the ground should be of lead or cast iron. The drains below the ground should be of either glazed socketed stoneware pipes, with cemented joints, or of cast-iron pipes, heavy water main strength, protected from rusting by the Bower-Barff process, or by coating with Dr. Angus Smith's solution. The iron pipes should have spigot and socket joints, run with molten lead and well caulked. Where large bodies of hot water pass through iron drains they should be fitted with expansion joints as may be necessary.

The jointing of stoneware pipes is shown in Fig. 782, that of iron pipes in Fig. 786.

Typical Drainage of a Small House (Figs. 787 to 792).—The sewage and refuse fluid matter of domestic buildings is conveyed from water closets, baths, sinks, etc., by means of a pipe to a cesspool or to a sewer. It is desirable to remove this matter as quickly as possible, before putrefaction takes place, and in such a manner that objectionable odours are reduced to a minimum and so that any adjacent soil is not contaminated. Impervious stoneware, or cast-iron pipes, and good water-sealed traps are used. It is desirable that all runs of pipes should be laid straight and an inspection chamber constructed at each horizontal junction to permit of easy access in case of any stoppage of the drain. The various branch drains of the sanitary system of a building should as far as possible be collected at a manhole, as shown in Fig. 789. Between the house drains and the sewer, or the cesspool, into which the drain discharges, there should be a disconnecting trap, as shown in Fig. 790, with an effective water seal to keep the house drains free from the sewer gases. All soil pipes should be taken to drains leading direct to a manhole without any intervening traps, and their upper ends should be continued above the highest window in the house, in order that they may serve as ventilators to the drains on the house side of the disconnecting traps. All waste pipes should discharge over or into open-trapped gullies, preferably the former, in order that all such pipes may be freely flushed


Fig. 787.
Fig. 792.
Scraper to clear cement from joints of pipes
Bath
Waste
W.C.
Soil Pipe.
Rain Water Pipe.
Gulley
R.W.P.
Front manhole
Disconnecting Trap.
Air Inlet
Road
Sewer
Section through front manhole and disconnecting trap.
Fig. 788.
Channel bends
Fig. 789
½ channels
sides of manhole in two half-brick thicknesses in cement
Air Inlet
Fig. 790.
Rendered in Cement
Bolster in cement half channel
Fig. 791.
Section through gulley

Figs. 787– 792

with air other than that from the drain. The laying of the drains, if of stoneware, is usually executed by the bricklayer, if of iron by the plumber. The drains must be laid so as to obtain a velocity of the liquids conveyed which will make the drains self-cleansing. This velocity is, in the case of house drains, usually secured by adopting the fall for

4 in. drains of 1 in 40.
6 in. drains of 1 in 60.
9 in. drains of 1 in 80.
12 in. drains of 1 in 100.

The following would be the method adopted for a small terrace house, as shown on plan (Fig. 787). All other systems would be laid on similar principles, but modified to comply with any peculiar conditions.

In the case of a terrace house, the drains should be laid after the building has been erected and the ground has had time to settle, or special provision made to support and keep the weight of the building clear of the drains.

The line of drains should be set out, the trenches cut to the true gradients, and the earth for the manholes removed; the sites of the manholes are then covered with 6 inches of good concrete and the drain trenches with 3 inches of concrete. The disconnecting trap should then be embedded in concrete in the lower manhole and connected with a drain leading to the sewage system. Where the roads have been taken over by the local authority, the connection from intercepting trap to sewer will usually be made by the authority's workmen at the expense of the builder. The invert channels in the lower manhole should then be bedded, and the straight portion of pipe between the two manholes laid and connected with the channels in the respective manholes. The pipes should be laid on invert blocks so that adequate space is available for making the joints. The various branch channel connections are then made, and drains laid to receive the waste water from sinks, baths, and rain-water pipes through a gully trap. Such trap disconnects the drain from the outer air. Branch drains connected with soil pipes are not trapped, but the soil pipe is continued upwards as before stated to act as a vent. The manholes are then built, the walls of which are 9 inches

thick and built in two separate half-brick thicknesses in cement mortar to render them less likely to leak. By having no "through" joints, excepting those for bedding the bricks, the manholes are not so likely to leak when a water test is applied. The spaces between the channels and walls are benched to a height of at least 6 inches and rendered in cement with a trowelled surface. The sides of the manhole should also be rendered in cement and the manhole covered with a gastight cover. Reinforced concrete manholes 3 to 4 inches in thickness are just as efficient as those formed of brick and less expensive. The excavation for the manhole need only be made to the outside dimensions of the concrete walls. When the channels and branches have been bedded and the spaces between them filled up with concrete to the highest level of the branch connection, the box forming the mould is lowered into position. The reinforcements out of $\frac{1}{4}$ to $\frac{3}{8}$ inch steel bent to form rectangles are connected together by upright rods about a foot apart, the main reinforcements being placed at about a 6-inch pitch, are now slipped over the mould, the concrete is then poured in and well tamped. In due course the mould is removed and the inside rendered and the benchings about the branches are formed. After the cement has thoroughly set the drains should be tested, and, if they are found to be satisfactory, concrete is put in to complete a 6 inch bed and to back up the pipes to half their height. If the pipes are laid under a house, they should be completely enveloped in concrete all round the pipe for a thickness of 6 inches. The ground may then be filled in and lightly punned.

The requirements for good stoneware pipes are that they should be of good shape, even surface, internally free from cracks and perfectly glazed. For the best work, pipes which have been tested under hydraulic pressure are used. Many stoneware pipes are distorted, being slightly curved in their length. Any pipes deviating from straightness should be so arranged as to lie in a horizontal plane and thus avoid irregularity of gradient in the invert of the drain.

The pipes should be jointed in cement and sand 1 : 1. This mortar should be mixed in small quantities and not

used after the setting action has commenced. Before bedding the pipe the socket of the last laid pipe should have a layer of mortar bedded in it ; the spigot end of the next pipe is then inserted and care taken that it is perfectly concentric with the preceding pipe. The mortar should then be tucked in and pressed home in the socket with a piece of wood cut for that purpose, and thus ensure the joint being quite solid. The joint is then flushed up with the mortar. On inserting the pipe a large amount of the mortar first bedded will be squeezed into the interior of the pipe ; this surplus cement should be carefully scraped out, each time a joint is made, with the tool shown in Fig. 792, sometimes called a badger, to prevent any projections on the inside of the pipe.

Another method of procedure, where straight lengths of drains occur as between two manholes, is as follows : On the prepared bed of concrete lay all the pipes in each length or run, and with a crowbar or prize force all the spigot ends of the pipes into the sockets and hard up against the socket shoulders. Then commence at the lower end and bed the first pipe on the concrete ; then lay the second pipe, and ensure that it is concentric with the first pipe by packing it at the side with concrete. In a similar manner centre and bed all the remainder of the pipes. Ample clearance about the pipes should be left for the making of the joints. Then return to the lower end and commence to make the joints. Care should be taken to force the mortar well into the joints ; ram it in with a wood tool specially made for this purpose. Finally, flush the joint level with the face of the socket. Most workmen leave an external fillet of mortar, but this is not essential. The advantage of this method is that the joints can be made without causing the slightest movement in the pipes. After testing the pipes they are enclosed with concrete, as prescribed by the Bye-laws.

The most frequent cause of drains failing to remain watertight is the movement of the earth in which they are embedded, especially in the case of clay soils. To obviate this many patent joints have been introduced in which bituminous jointing material is adopted. This material remains plastic, and, consequently, some movement of the

pipes can occur without necessarily causing disruption of the joint. Some of the best known of these joints are Stanford's, Jennings' and Lowe's.

Disconnection of House Drains from Sewer.—The position for disconnecting the house drain from the sewer by means of a manhole and disconnecting trap must be "as near as may be practicable to the point at which such drain may be connected with the sewer."

Disconnection of Waste Pipes from Soil Drains.—The disconnection of bath, rain-water and sink waste pipes from sewage drains is effected by means of gully traps, shown in Fig. 791.

Intercepting Tank for retaining Petroleum.—In the case of drains collecting surface drainage which may be charged with petroleum, as in the case of petrol stores, garage yards, it is necessary to eliminate the petroleum before the drainage enters the sewer. In the London district this must be done by an intercepting tank, as shown in Fig. 793, consisting of three tanks, in each of which a submerged outlet arrests the flow of the oil.

Ventilation.—Ventilation of drains may, in order to comply with the Bye-laws, be provided in one of two ways:

(*a*) By an inlet near the trap disconnecting the drain from the sewer, and an upcast at a point of the drain as far distant as may be practicable from the inlet; or

(*b*) By reversing the positions of inlet and upcast.

The general form of the combination of air-inlet pipe drain and upcast ventilator pipe can in either case be compared to the letter J.

In the former case the air-inlet pipe fixed at the lower end of the drain is provided with an open grating or with a mica flap valve, which closes with any back pressure when it is desirable to prevent the escape of any air from the drains. The direction of the air current is from the lower to the higher end of the drain. The air in the longer air pipe, fixed at the higher end of the drain, will usually be somewhat higher in temperature than the external air and

consequently will be forced up by the heavier column of air which passes into the inlet pipe. The difference in weight of the two columns of air is so small that it is often counteracted by the sun heating and rarefying the air in the inlet

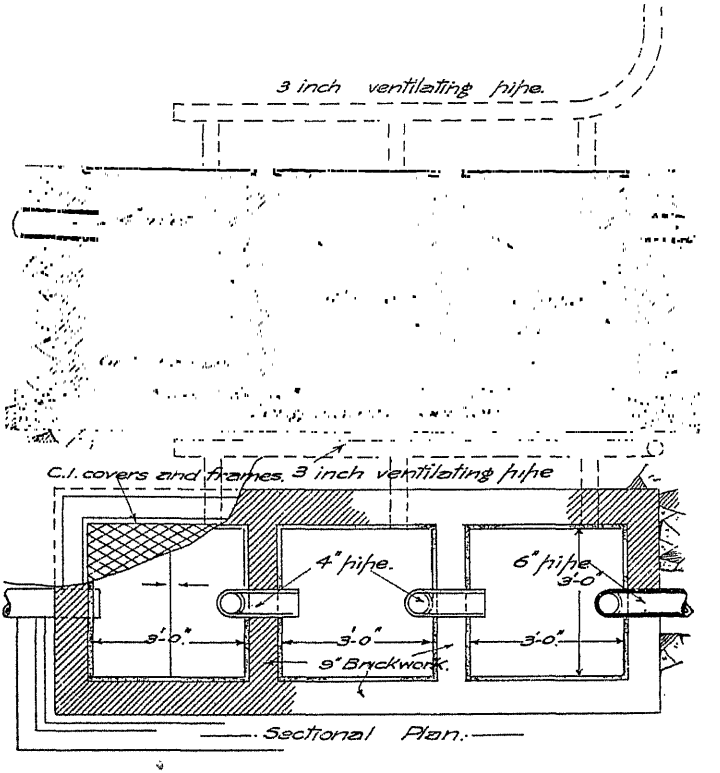


Fig. 793.

pipe. A gust of wind blowing down the ventilating outlet pipe, and particularly the continual flow of waste water from the fittings (which are usually at the back of the house) towards the sewer, will retard, drive back, or reverse the air current. This leads to the suggestion that in some cases it is advisable to so arrange the system of ventilation

as specified in the alternative (*b*), so that the longer upcast pipe would be at the lower end of the drain and the shorter inlet pipe at the higher end of the drain ; the flow of sewage would then accelerate instead of retard the air current.

Anti-Syphonage Pipes.—Owing to various causes the water in traps may be forced or sucked out. These causes may be—

(*a*) A partial vacuum caused by water passing the junction of a pipe.

(*b*) Pressure resulting from water approaching such a junction.

(*c*) The momentum of discharge of the water through the trap.

(*d*) Syphonage in the trap.

(*e*) Evaporation.

To prevent the loss of the safeguard afforded by the water seal, all traps inside buildings should be provided with special air supply pipes, as shown in Figs. 797 to 800. Such pipes provide for a continuous current of air to pass through and ventilate the pipes and drains when the sanitary fittings are not being used, and also to keep the air pressures equal on both sides of the water seal of the traps when the fittings are being used, and thus prevent the traps being unsealed from the causes stated (*a*), (*b*), (*d*).

In the case of W.C.'s and ranges of lavatory basins the anti-syphon pipes will be branched into a main pipe, but in the case of isolated lavatories on external walls the pipes are only taken through the wall and left with open ends. They are then often termed " puff " pipes.

Manholes or Inspection Chambers.—Inspection chambers, or manholes, should be constructed at all points of junction of branch drains into the main drain, and also for access to the main and branch drains for examination and cleansing, or for removing obstructions.

The chambers are built with brick-in-cement walls on concrete foundations, and made water-tight. This may be done by rendering them inside with Portland cement and clean, sharp, washed sand. The internal dimensions should be sufficient to permit ready access for cleansing or clearing

the drains, and will vary according to the number of branch drains connected. The depth will be dependent upon the depth of the drain. For shallow manholes a width of

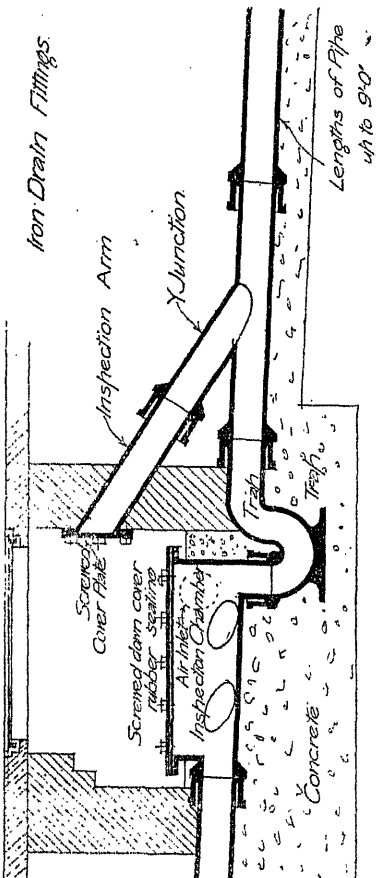


Fig. 794.

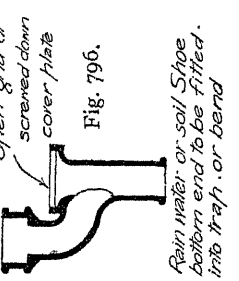


Fig. 796.

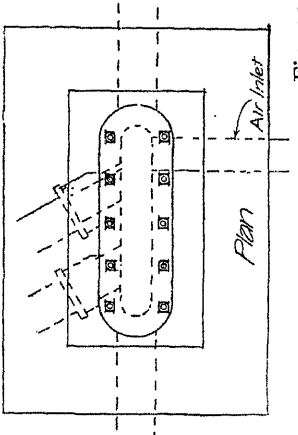


Fig. 795

Figs. 794—796.

2 feet and a length of 9 inches for each branch on one side, with a minimum of 2 feet, will usually suffice. Fig. 790 is a section of a manhole and disconnecting trap.

The manholes should be provided with air-tight covers,

and in some cases should be ventilated by means of separate pipes, the openings of which should be clear of all doors, windows and other openings into the house.

Figs. 794 to 796 show the construction of a manhole arranged to receive cast iron pipes and fittings.

W.C. Rooms.—Rooms containing water closets should be well ventilated by openings through the external walls, and the walls and floors should be so constructed as to prevent any local smells passing into the house.

The Bye-laws require that one of the sides at least shall be an external wall abutting upon an open space or area having a minimum area of 100 square feet except in the case of W.C.'s below ground, which must abut similarly upon an area of at least 40 square feet.

The closet must have a window of which an area of at least 2 square feet shall open directly into the external air, and in addition permanent ventilation, by means of, *e.g.*, an air brick, must be provided.

The Bye-laws also prescribe that solid walls shall be interposed between water closets and any living room, workplace, etc., and it is of course necessary that similar provision should be made in respect of floor and ceiling.

No water closet shall be approached directly from any living room, food store or workplace, and in important cases, such as hospitals, public buildings, etc., the further precaution should be taken to isolate the water closets by well-ventilated lobbies.

Water Closets.—The closet pan consists of a trapped bowl or basin to receive the fæcal matter, which is carried away, by the use of water, into the drains and sewers.

There are many kinds of water closets on the market; but there are three types now generally recognised as being efficient from an hygienic standpoint: (1) The wash down (Fig. 797); (2) the syphonic (Fig. 799); and (3) the valve (Fig. 800).

A good water-closet pan should possess the following properties: (1) A large water area to prevent the basin being fouled; (2) a good water seal, not less than 2 inches, to prevent the escape of sewer gases; (3) a flush of water

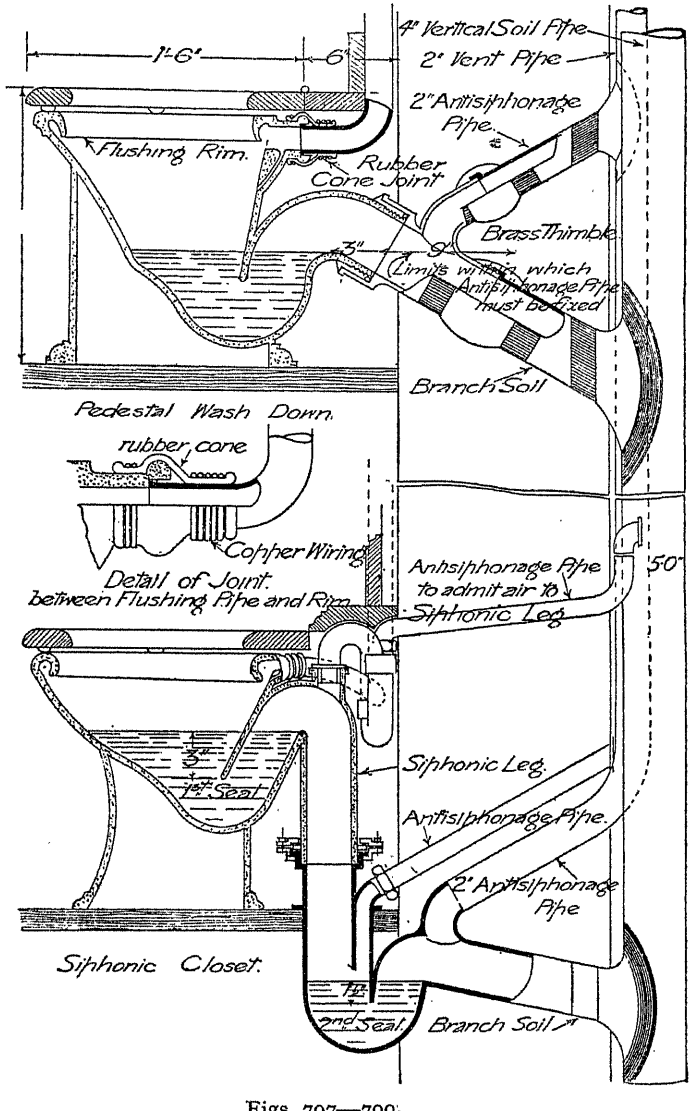


Fig. 797.

Fig. 798.

Fig. 799.

Figs. 797—799.

sufficient to entirely remove the contents of the trap and to refill it with fresh water; (4) provision in the branch soil pipe to prevent syphonage; (5) no hidden mechanism requiring enclosed and unventilated places; (6) to be of a glazed material to ensure facility in cleansing; (7) the joint to soil branch pipe should be readily accessible.

Fig. 797 shows the general arrangements of a wash-down closet, with the connection to soil and anti-syphonage pipe, also the flushing pipe and flushing rim. The simplicity of the wash-down renders it an excellent apparatus for cottages, public conveniences and institutions. The flush, however, is frequently insufficient to entirely remove the contents of the basin. This may be due to faulty design of the pan or inadequate fall from the flushing cistern. Fig. 798 shows an enlarged detail of the joint between the basin and the lead flushing pipe. The jointing material is putty; the joint is covered with an indiarubber canvas-lined cone, which is secured to the rim and pipe with copper wire. The joint at the foot of the soil pipe with the drain is shown in Figs. 783 to 785 for iron and stoneware drains respectively.

Fig. 799 shows a syphonic closet. The advantages claimed for this apparatus are (1) an extra deep seal; (2) a second seal; and (3) the certain removal of the contents of the basin at each discharge from the flushing cistern. The action is as follows: when the flush is made a spray first falls into the syphonic leg, causing a partial vacuum; this causes motion in the contents of the first trap; the momentum is increased as the action continues until the whole of the contents are discharged with considerable velocity down the syphonic leg and through the second seal. The water flowing through the flushing pipe then recharges the first trap. Fig. 799 shows all the details of the connections with the soil and ventilation pipes.

The valve closet shown in Fig. 800 is obsolete although there are many still in use, and it is efficient.

A disadvantage is the amount of the mechanical arrangements existing that must be covered up. This, however, has been simplified and perfected to such an extent, both with respect to its efficiency and the space occupied, that

this consideration is of little account, and the whole can be enclosed in a porcelain covering occupying a space very

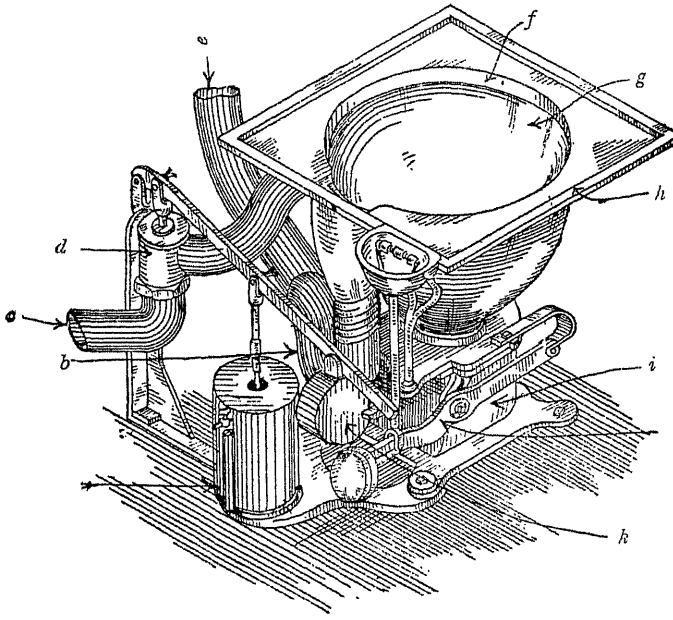


Fig. 800.

a Air cylinder to regulate flush.
b Overflow trap.
c Service pipe.
d Supply valve.
e Vent pipe to valve box.
f Flushing rim.
g Earthenware pan.
h Table top.
i Valve box.
j Lever for opening and closing basin valve.
k Cast-iron weight for shutting supply valve.

little larger than an ordinary wash-down. It is expensive.

Flushing Cisterns.—Flushing cisterns should be fixed over every water-closet and slop sink to cleanse them after being used.

For domestic use flushing cisterns or "water waste-preventors" are usually of one of the following types: Valve-syphon, displacer-syphon or valve. The latter is

Fig. 801. Fig. 802.

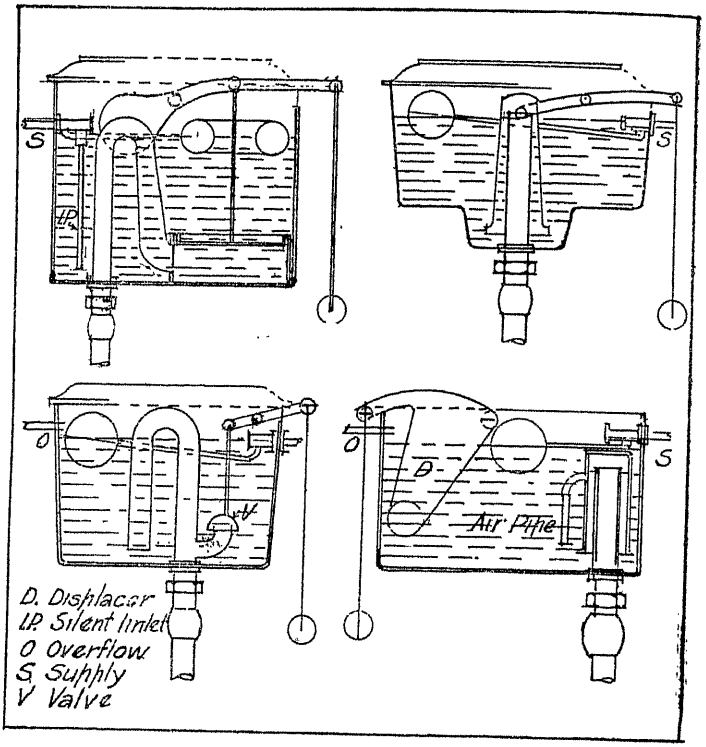
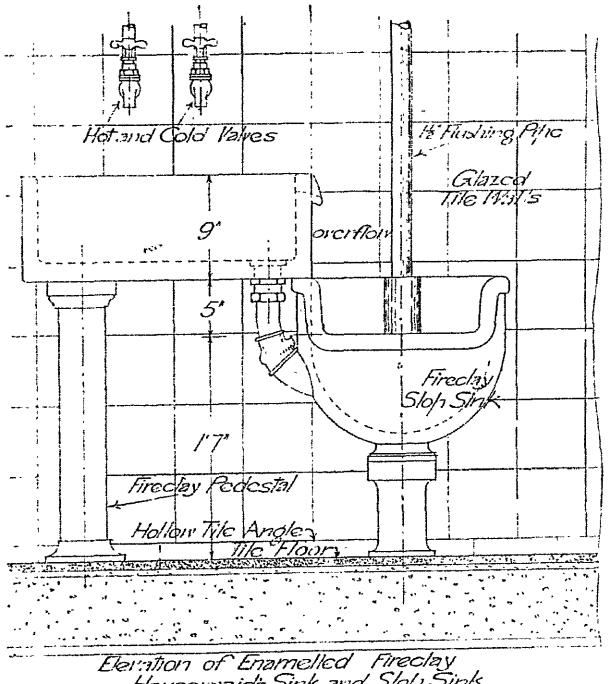


Fig. 803. Fig. 804.

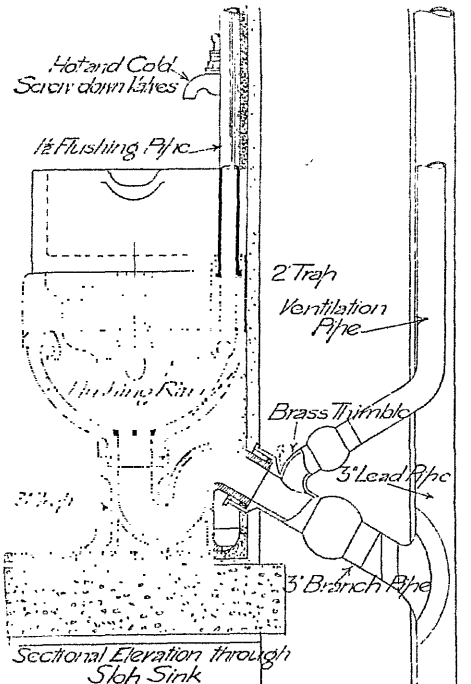
Figs. 801—804.

generally not permitted by the water companies, as it is uneconomical in the use of water. Figs. 801 to 804 show the various types, Fig. 803 being a valve-syphon, the remainder displacer syphons.



Elevation of Enamelled Fireclay Housemaid's Sink and Slop Sink

Fig. 805.



Sectional Elevation through Slop Sink

Fig. 806.

Figs. 805—806.

The urinals and drains of public buildings, workshops, and large houses are usually flushed by cisterns which act automatically at regulated intervals of time. The cisterns are fitted with annular syphons. The syphon consists of an inner leg, the higher end of which is in a reservoir tank and the lower end in a small under-chamber, the latter being constructed so as to retain enough water to cover the lower end of the pipe. The inner leg is covered by an outer pipe, which has a closed top and an open bottom end. This pipe forms a cap over the inner leg, and leaves an annular space through which the water can rise and flow into the top end of the inner leg. The water supply is regulated by a tap arranged to flow either quickly or gently into the tank as may be desired. When the tank has filled so that the water is an inch or two above the cap of the syphon, the attained head is sufficient to compress the air inside the syphon so as to force away the water in the bottom of the inner leg. As soon as the water is forced away the air escapes, and the inner leg is quickly filled with water, which flows away with a good cleansing force into the drains. When the tank has been emptied it slowly refills, and the whole of the actions are repeated.

This appliance is used also for collecting driblets of rain or waste water, and storing it until the tank is sufficiently filled to discharge automatically into the drains. By this arrangement the drains are kept much cleaner than they would be if the small quantities of waste water simply dribbled into them.

Housemaids' Sinks.—In hotels and all large residences having a considerable number of bedrooms, a housemaid's closet containing a washing-up sink, with hot and cold services, and a slop sink, are essential for convenient and hygienic working. Every care should be taken to ensure facility in cleansing; the walls and the floor should be tiled, and the two sinks formed and fitted as shown in Figs. 805 and 806. It will be noted that the waste and overflow of the washing sink discharge into the slop sink, thus giving the maximum of flushing for the latter. The waste of a slop sink must be constructed similarly to a soil pipe.

Fig. 807.

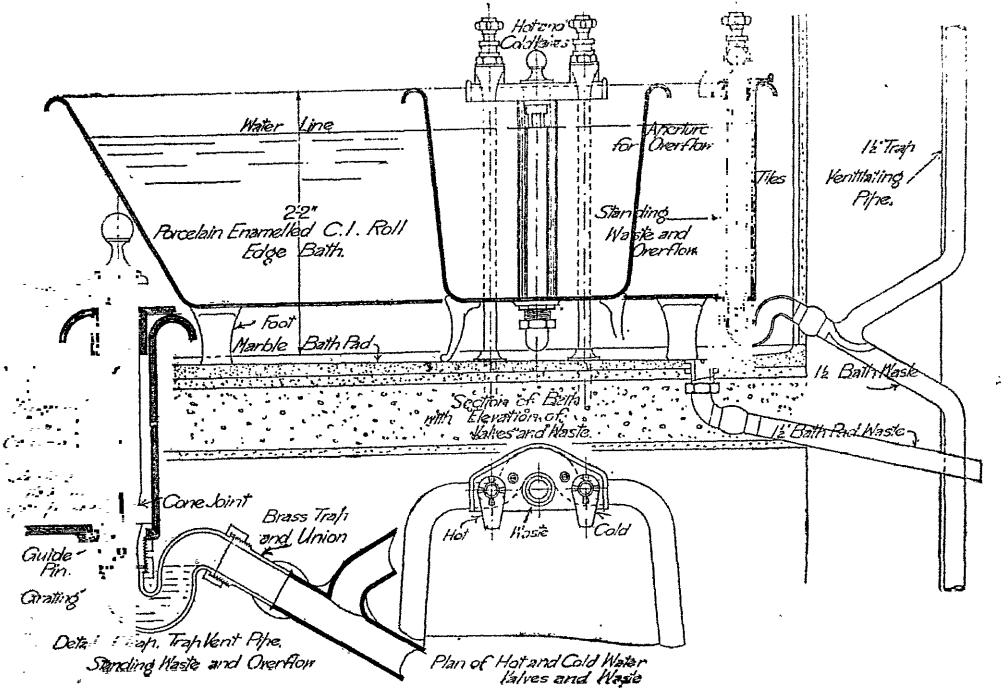


Fig. 809.

Fig. 808.

Figs. 807—809.

Baths.—Figs. 807 to 809 show the arrangements of supply and waste and the fitting of a good class bath. Baths are now generally made of porcelain or cast-iron porcelain enamelled. The former are excellent from an hygienic standpoint, but they absorb a considerable amount of heat from the water, and their weight is a disadvantage which frequently causes a special construction necessary. Heavy cast-iron porcelain enamelled baths are preferable, as in these the above objections are minimized. Bathrooms should be fitted with impervious flooring and tiled walls to ensure facility in cleansing. The bath should be placed in a lead safe to carry off any water that may fall over the sides. A dished marble pad, as shown in Fig. 807, is usually fitted instead in good work. The hot and cold water services and the waste should be so placed that they can be easily seen and attended to. Fig. 808 shows a standing waste and overflow. This type is the most modern, and, as the waste pipe can be removed for cleansing purposes, is preferable on hygienic grounds, but it is objected to under the water supply regulations which require that all overflows shall act as "warning pipes." From the illustration it will be seen that overflowing can continue indefinitely without being observed.

Lavatory Basins.—Figs. 810 to 813 illustrate a lavatory basin with hot and cold services and waste and overflow pipes. The foregoing general remarks concerning baths also apply to lavatory basins. In the example illustrated a different method of overflow arrangement is shown.

All overflow pipes must discharge in the open air, and must be easily and distinctly visible to comply with the Bye-laws.

Waste Pipes.—See the L.C.C. Drainage Byelaws for Waste-water Fitments. The waste pipes of all sanitary appliances when fixed inside houses should be trapped immediately under the fittings. Drawn lead traps are now generally employed for wastes (see B.S.S. 504 for types, weights and other properties of lead traps and the tests for the same. The traps should be provided with screw caps for cleansing purposes.

Fig. 810.

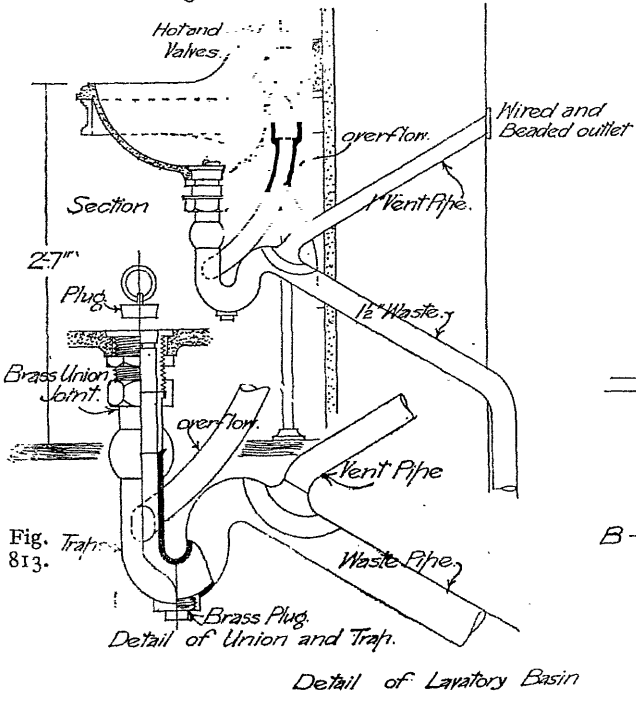
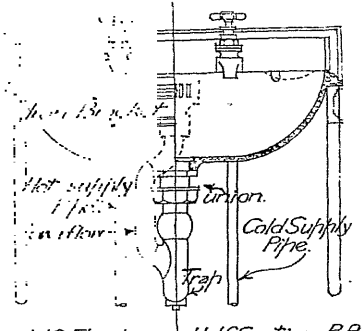


Fig. 811.



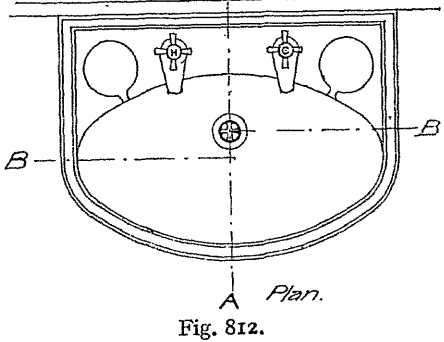


Fig. 812.

Figs. 810—813.

In the case of fittings of which the wastes discharge into open hoppers or gullies immediately outside, when the waste pipe does not exceed, say, 6 feet, the provision of such ventilating or " puff " pipes is sometimes omitted.

Joints of Pipes.—The joint of two stoneware pipes is made by rings of gasket pressed well against the shoulder of the socket, and the latter filled up with Portland cement, as shown in Fig. 782.

Fig. 784 shows the connection of an iron with a stoneware drain. The joint is made by gasket pressed in the socket against the spigot and Portland cement filling up the socket.

Fig. 785 shows lead with a stoneware drain. The joint is made by a brass thimble with a wiped soldered joint to the lead, the lead pipe and brass thimble are both flanged as shown, gasket and Portland cement making the connection as before.

Fig. 783 shows lead with an iron drain. In this case the jointing materials of the joint differ from that previously shown by being of gasket and metallic lead well caulked.

Fig. 786 shows the connection of iron to iron drain, the joint being made by gasket pressed against the spigot end of the pipe and the socket filled up with a ring of metallic lead well caulked.

Fig. 814 shows a joint between an iron and a stoneware pipe. A wrought iron or brass flange in two pieces is bolted to the socket end of the cast-iron pipe, a corresponding flange in one piece is passed over the stoneware pipe, the jointing material is bedded and an indiarubber ring is placed between the second flange and the socket rim of the cast-iron pipe. The two flanges are then bolted together, the pressure causing the rubber packing to come into close contact with both the stoneware and iron pipes.

Fig. 817 shows a similar arrangement for a brass tail piece to a stoneware pipe.

Fig. 815 shows a flanged joint for a cast-iron steam pipe. The faces of the flanges are planed, a ring of asbestos packing is inserted, and the two surfaces are compressed by the two bolts shown.

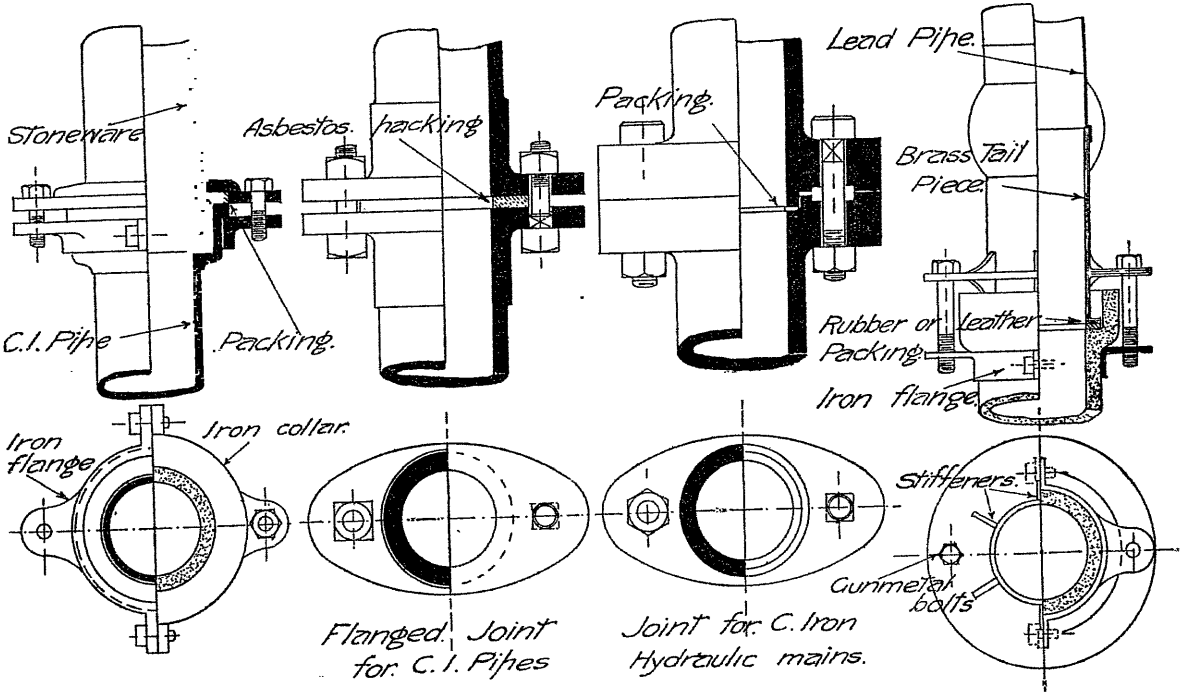


Fig. 814. Fig. 815. Fig. 816. Fig. 817.

Figs. 814—817.

Fig. 816 shows a flanged joint for a cast-iron hydraulic main. It differs in principle from the preceding in that it has a tongued and rebated joint.

Where security against leakage under great pressure is necessary the end of one pipe is turned with a flat face and the other with a sharp V-shaped edge. The two pipes are then drawn together by flanges or by coupling pieces having right- and left-handed screws.

See B.S.S. 416 for the forms and properties of cast-iron, soil, waste, ventilating and rain-water pipes.

Drain Testing.—The soundness of drains and fittings may be tested by the following methods: (1) essences or volatile oils; (2) smoke; (3) hydraulic pressure; (4) air pressure.

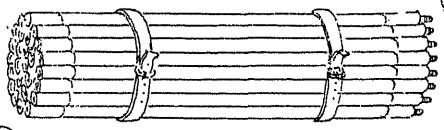
1. Defects may be discovered by carefully pouring volatile oil, such as strong essence of peppermint, followed by hot water, down the highest part of the drainage system. If the odour of the oil escapes, the existence of a defect is proved. This test is not conclusive, as a defect may not be discovered by its application. It is, however, useful in making preliminary surveys of existing systems.

2. The second test consists in charging the drains with smoke by means of a smoke rocket or a smoke machine. Only very pronounced defects are likely to be disclosed by this test if applied by a rocket which develops little pressure to cause the smoke to escape through any defect; a machine should always be used. The machine (see Fig. 818) consists principally of two parts, a container in which material such as oily cotton waste is burnt, giving a heavy smoke with a well-marked smell, and a bellows driving a current of air through the container into the drains. The container is water-jacketed and the pressure resulting from the pumping is limited to that which would be resisted by the depth of the water. This test is very effective when skilfully used and can be applied through the fresh air inlet to the drains, or through the ventilating pipe, or through a gully trap from which the water has been removed. An escape of smoke at any part would indicate a defect.

3. The hydraulic pressure test consists in stopping up the drain by means of an inflated india-rubber bag

Fig. 821.

Drain Rods and Fittings



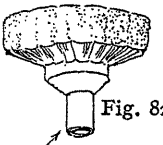


Fig. 822.

Brush.

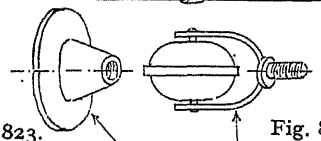


Fig. 823.

Rubber Flange.

Fig. 824.

Wheel.



Fig. 825.

Jag.

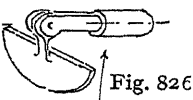


Fig. 826.

Scraper.

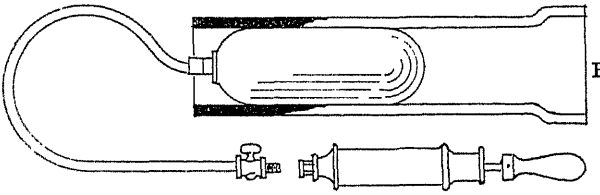


Fig. 819.

Bag Drain Stopper and Pump.

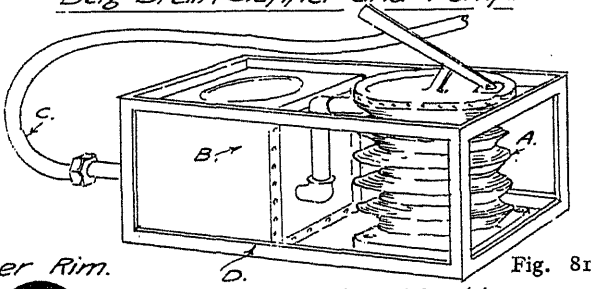


Fig. 818.

Smoke Machine.

A. Bellows.
B. Tank and Smoke-box.
C. Flexible hose.
D. Frame container.

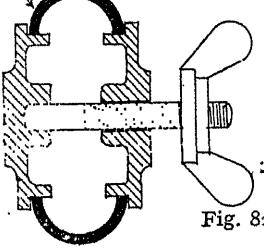


Fig. 820.

Figs. 818—826.

(Fig. 819), or by an india-rubber plug (Fig. 820), and then filling the whole of the drainage system with water until it rises to the level of a surface gully or another manhole. If after a time, say, half an hour, the water is found to subside, this would denote a leakage. This test is the only reliable one for drains, and is usually applied to drains when laid underground. For the vertical parts of a drainage system, such as soil pipes, etc., the smoke test is used and considered sufficient. All drains and manholes should be subjected to the hydraulic and the work above ground to the smoke test before being covered in and on completion of the works. In the case of extensive drainage systems it will be advisable to test the drains in sections; not more than 6 feet head should usually be applied to any part of a drain; a greater pressure, especially when testing old drainage systems, may develop defects not previously existing.

4. The air pressure test as applied to drains consists of plugging the soil pipes, and ventilating pipes, if any, then examining if all manhole covers are properly fitted and all traps are properly sealed with water. Air is then forced into the drains by means of a pump similar to a large size bicycle pump. A glass syphon pressure gauge filled with water is necessary to show the air pressure exerted in the drains. Under ordinary conditions an air pressure of 1·5 inch can be exerted. If a test under a greater pressure than 1·5 inch is desired it becomes necessary to plug all the gully and water closet traps in addition to the main drain trap in the disconnecting manhole. The syphon pressure gauge should be of a depth in proportion to the air pressure to be resisted.

Figs. 818 to 826 show draw rods and apparatus for unstopping and cleaning drains.

The Bye-laws of the London County Council must be complied with in London, and may be followed with advantage elsewhere.

Bye-laws* made by the London County Council on May 20th, 1930, under Section 39 (1) of the Public Health (London) Act, 1891, and Section 24 of the London County

* NOTE.—The footnotes on pp. 930 and 931 do not form part of the bye-laws.

Council (General Powers) Act, 1928, with respect to water-closets, urinals, earth-closets, privies, and cesspools, and the proper accessories thereof in connection with buildings.

Definition.

1.—In these bye-laws, unless the context otherwise requires :—

" Work," means the construction, partial or entire reconstruction, alteration, or repair of any water-closet, urinal, earth-closet, privy, cesspool, in connection with a building, or the fitting or fixing in or in connection with any such water-closet, urinal, earth-closet, privy, cesspool, of any pan, basin, trap, pipe, apparatus or accessory thereof.

" Occupied building " means a house or part of a house which is let to, or occupied by, members of more than one family, but does not include a common lodging-house.

" Public room " means a room used as a place of public worship, a place of public entertainment, a public lecture room, public exhibition room, place of public assembly, or a place used for any other public purpose.

" Sanitary Authority " means the Council of the Metropolitan Borough in which any work within the meaning of the bye-laws has been, is being or is about to be executed.

" Builder " means the builder, contractor or person actually executing any work, and for the purpose of bye-law No. 9 relating to the giving of notices, means the builder, contractor or person intending actually to execute such work, but does not include a workman in the employ of such builder, contractor or person, and where after written request by the Sanitary Authority the owner or occupier of any premises on which any work has been or is being executed fails to furnish such authority with the name and address of such builder, contractor or person includes such owner or occupier.

" Owner " means the owner within the meaning of the Public Health (London) Act, 1891, but where the premises are, without his consent in writing, or connivance, let or sub-let by some other person, the person so letting or sub-letting shall be deemed to be the owner to the exclusion of the first-mentioned owner.

" Landlord " means the person (whatever may be the nature or extent of his interest in the premises) by whom or on whose behalf a building or part of a building is let, or who for the time being receives or is entitled to receive the profits arising from such letting.

" Occupier " means a person to whom a building or part of a building is let for his use or occupation.

Water-closets.

2.—A builder constructing a water-closet in connection with a building shall comply with the following requirements :—

(1) *Situation.*—Such water-closet shall be so situated that at least one of its sides shall be an external wall, which shall be in conformity with the following conditions :—

(*a*) If the water-closet is so situated that the surface of the

floor is at or above or not more than five feet below the ground level such external wall shall abut immediately upon :—

(i) A street ; the surface of which shall be at a level not exceeding five feet above the level of the floor of the water-closet, or

(ii) An open space dedicated to the public or permanently secured to the building in connection with which the water-closet is provided with a surface area of not less than one hundred square feet measured horizontally at a level not exceeding five feet above the level of the floor of the water-closet and a minimum width of three feet if enclosed on not more than two sides and seven feet if enclosed on every side.

(*b*) If the water-closet is so situated that the surface of the floor is more than five feet below the ground level, such external wall shall abut upon an area or open space permanently secured to the building in connection with which the water-closet is provided, not covered in otherwise than by a suitable grating, and having a minimum horizontal superficial area of forty square feet and a minimum width of five feet or, where such area or open space is restricted to lighting and ventilating the water-closet, measuring horizontally not less than five feet by five feet. Such area or open space shall immediately abut upon, and the surface of such area or open space shall be not more than twelve feet below, a street, a forecourt immediately adjoining a street, or an open space as hereinbefore prescribed in paragraph (*a*) (ii).

Provided always :—

(i) That where the water-closet does not exceed twenty feet in height or the height of the storey in which the water-closet is situated the water-closet may be so situated as not to have an external wall if a street or an open space as hereinbefore prescribed in paragraph (*a*) is available at the level of and abutting on the roof of the water-closet.

(ii) That the water-closet may be so situated that none of its sides is an external wall and that an open space as hereinbefore described in paragraphs (*a*) or (*b*) need not be provided if means of artificial lighting and a system of mechanical ventilation are furnished to such water-closet in accordance with paragraph (4) (*e*) of this bye-law.

(2) *Entrance and entrance lobby.*—Such water-closet shall not be situated within nor entered from any room used for human habitation, or as a scullery, schoolroom, office, factory, workshop, workplace, or for the manufacture, preparation, storage or sale of food or drink for man, or as a public room, except through the external air or an intervening entrance lobby.

The entrance lobby shall be constructed of solid and suitable materials so as to secure aerial disconnection between such water-closet and any room specified in the foregoing paragraph of this bye-law, and such lobby shall be :—

(*a*) Provided with close-fitting and self-closing doors.

(*b*) Adequately lighted and ventilated.

Provided always that a water-closet used exclusively with a bedroom or dressing-room may be entered directly from such room.

(3) *Form of construction.*—Such water-closet shall :—

(*a*) If situated wholly or partly within a building be :—

(i) Properly ceiled.

(ii) Enclosed by solid walls or partitions of brick, concrete, or other suitable material, and every such wall or partition shall, where abutting upon any room specified in paragraph (2) of this bye-law, be finished with an impervious surface towards the water-closet.

(*b*) If not situated wholly or partly within a building be :—

(i) Constructed of brick, concrete, or other suitable material, with any wall abutting upon any room specified in paragraph (2) of this bye-law finished with an impervious surface towards the water-closet.

(ii) Covered with a properly constructed roof provided with suitable means for discharging rain-water therefrom.

(*c*) Where entered directly from the external air be paved with hard and impervious material laid to a suitable fall and finished with a smooth surface at least three inches above the level of the street or area or open space from which it is entered.

(*d*) Be provided with a proper entrance door and fastenings.

(4) *Lighting and ventilation.*—Such water-closet shall :—

(*a*) If an external wall is provided, and if such water-closet contains a single soil-pan or basin only, have :—

(i) In such external wall a suitable window of an area exclusive of the frame of not less than two square feet, of which window an area of at least half shall open.

(ii) An air-brick, air-shaft, or other adequate means of constant ventilation.

(*b*) If an external wall is provided, and if such water-closet contains two or more soil-pans or basins, have :—

(i) In such external wall a suitable window or windows of a total area exclusive of the frame or frames of not less than one-fifth of the floor space, of which window or windows and area of at least half shall open.

(ii) One or more air-bricks, air-shafts, or other adequate means of constant ventilation, with a total unobstructed sectional area of not less than twenty square inches per soil-pan or basin, in a suitable position or positions.

(*c*) If not provided with an external wall but provided with an open space as prescribed by paragraph (1) (*a*) and proviso (i) to paragraph (1) of this bye-law have means of lighting and ventilation in the form of :—

(i) A suitable glazed lantern light or skylight of an area exclusive of the frame of not less than one-fifth of the floor space with louvred or other suitable openings equal to at least one-tenth of the floor space.

(ii) One or more air-shafts or other adequate means of constant ventilation, with a total unobstructed sectional

area of not less than twenty square inches per soil-pan or basin, in a suitable position or positions.

Provided always that such water-closet, if situated under a public footpath where the provision of a lantern light or skylight for lighting and ventilation is, for such reason, impracticable, shall have :—

(i) Means of lighting in the form of a pavement or other similar light of equal area to that hereinbefore prescribed in paragraph (*c*) (i) for a lantern light or skylight.

(ii) Means of ventilation in the form of suitable inlet and outlet air-shafts, the outlet shaft being fitted with a suitable exhaust fan.

(*d*) If aerially disconnected from all buildings occupied by any person and directly approached from the external air, have means of lighting and ventilation as prescribed in either of the foregoing paragraphs (*a*), (*b*) or (*c*), or other equally effectual means.

(*e*) If so situated that none of its sides is an external wall and an open space is not provided, have suitable means of artificial lighting and a suitable system of mechanical ventilation complying with the following requirements :—

(i) The system of ventilation shall be separate and distinct from any system of ventilation installed for any other purpose.

(ii) A sufficient number of suitably placed fresh-air inlets shall be provided.

(iii) The fan and motor or other mechanical means shall be in duplicate and capable of extracting air from the water-closet at a minimum rate of 750 cubic feet per hour per soil-pan or basin. The aerial content of the water-closet shall be changed at least three times per hour.

(5) *Materials, form of construction and trapping of soil-pan or basin.*—Such water-closet shall be furnished with a suitable soil-pan or basin :—

(*a*) Constructed of glazed earthenware, enamelled fireclay, or other equally suitable material.

(*b*) Fitted with a flushing rim.

(*c*) Of such shape, capacity and mode of construction as to receive and contain a sufficient quantity of water to cover the filth which may be deposited in such soil-pan or basin and to allow all such filth to fall free of the sides thereof and directly into the water.

(*d*) Provided with a suitable and efficient trap * :—

(i) Constructed of lead, copper, cast-iron, glazed earthenware, enamelled fireclay, or other equally suitable material,

* In addition to the requirements here enumerated, a trap must conform with bye-law No. 5 (12) made under [illegible] Management Act, 1855, and the Metropolis Management Acts Amendment (Bye-laws) Act, 1899, in respect of the water seal.

with an exposed and accessible outgo or outlet for connecting to a soil pipe or drain.

(ii) Fixed immediately beneath such soil-pan or basin.

(*e*) Furnished, except in the case of an Eastern fitment (siege turque), with suitable seat rims or insets or a hinged seat.

(6) *Flushing cistern and apparatus.*—Such water-closet shall be provided with a suitable flushing cistern for the purpose of cleansing such soil-pan or basin and such cistern shall be so constructed, fitted, placed and supplied as to comply with the following requirements :—

(*a*) It shall be separate and distinct from any cistern used for drinking water.

(*b*) The discharging or flushing capacity shall not be less than two gallons of water.*

(*c*) It shall be fitted with :—

(i) A ball-valve so arranged as to re-fill the cistern with water within a period not exceeding two minutes after the cistern is operated.

(ii) A suitable apparatus for the effectual application of water to and the effectual cleansing of such soil-pan or basin and for effecting the prompt removal therefrom and from the trap connected therewith of any solid or liquid filth which may be deposited therein.

(iii) A flush pipe constructed of lead, copper, iron, or other equally suitable material connected to the cistern and to the flushing rim of the soil-pan or basin by a union or other equally suitable form of connection and having throughout an internal diameter of not less than one inch and a quarter.

Provided always :—

(i) That there shall not be any direct connection between any water-service pipe upon the premises and any part of the soil-pan or basin.

(ii) That where the water for flushing purposes is obtained from a private well or other source having no connection with the mains of a statutory water authority the requirement relating to the provision of a flushing cistern shall be deemed to be complied with in any case where the soil-pan or basin is connected with an effectual flushing valve supplied with water from a storage cistern of adequate capacity used solely for the purpose of flushing water-closets, slop sinks or urinals, and complying in all other respects with the foregoing paragraphs (*b*), (*c*) (ii) and (iii).

* The flushing capacity here required is the maximum flush prescribed by "Regulations made under the Metropolis Water Act, 1871." These Regulations further prescribe that no alterations shall be made in any fittings in connection with the supply of water by the Metropolitan Water Board without two days' previous notice in writing to the Board.

(7) For the purposes of this bye-law a compartment containing two or more soil-pans or basins separated by partitions shall be deemed to be two or more water-closets, unless the partitions do not exceed seven feet in height and have a space of at least six inches between the tops and bottoms of such partitions and the ceiling and floor respectively.

Urinals.

3.—A builder constructing a urinal in connection with a building shall comply with the following requirements :—

(1) *Situation, entrance and entrance lobby, form of construction, lighting and ventilation.*—Such urinal shall comply with the requirements contained in paragraphs (1), (2), (3) and (4) of the preceding bye-law No. 2 for which purpose it shall be deemed to be a water-closet.

Provided always that the fan and motor or other mechanical means described in paragraph (4) (*e*) (iii) of bye-law No. 2 shall be capable of extracting air from the urinal at a minimum rate of 750 cubic feet per hour per urinal basin, or each width or length not exceeding two feet three inches of stall or trough respectively, and that the aerial content of the urinal shall be changed at least three times per hour.

(2) *Materials, form of construction and flushing of a urinal basin, stall or trough.*—Such urinal shall be provided with :—

(*a*) A basin, stall or trough constructed of glazed stoneware, glazed earthenware, enamelled fireclay or other equally suitable material of such shape as will facilitate maintenance in a state of cleanliness.

(*b*) A suitable flushing cistern so constructed, fitted, placed and supplied that :—

(i) It shall be separate and distinct from any cistern used for drinking-water.

(ii) The discharging or flushing capacity shall not be less than one gallon of water for each connected basin, or each width or length not exceeding two feet three inches of stall or trough respectively.

(iii) It shall be capable of being filled or charged with water within a period not exceeding twenty-five minutes or such less period as will permit, while the urinal is in use or available for use, a flushing operation of sufficient frequency to ensure the maintenance of such basin, stall or trough in a state of cleanliness.

(iv) It shall be fitted with a suitable automatic discharging apparatus connected to the urinal basin, stall or trough by an adequate flush pipe or pipes of lead, copper, iron, or other equally suitable material having a minimum internal diameter of half an inch and fitted with a suitable spreader or sparge pipe so as effectually to distribute the water over the internal surface of every basin, stall or trough.

Provided always :—

(i) That there shall not be any direct connection between any water-service pipe upon the premises and any part of a urinal basin, stall or trough.

(ii) That where the water for flushing purposes is obtained from a private well or other source having no connection with the mains of a statutory water authority, the requirement relating to the provision of a flushing cistern shall be deemed to be complied with in any case where the urinal basin, stall or trough is connected with an effectual flushing valve supplied with water from a storage cistern of adequate capacity used solely for the purpose of flushing water-closets, slop sinks or urinals, and complying in all other respects with the foregoing paragraphs (*b*) (ii), (iii) and (iv).

Earth-closets.

4.—A builder constructing an earth-closet in connection with a building shall comply with the following requirements :—

(1) *Situation.*—Such earth-closet shall be so situated that at least two of its sides shall be external walls, which walls shall abut immediately upon a street, or upon an open space as prescribed in bye-law No. 2 (1) (*a*) (ii) for a water-closet, but with the surface of such street or open space measured horizontally at a level of at least three inches below the level of the floor of such earth-closet.

(2) *Entrance.*—Such earth-closet shall be entered directly from the external air.

(3) *Form of construction.*—Such earth-closet shall :—

(*a*) If situated wholly or partly within a building be constructed in accordance with the requirements contained in bye-law No. 2 (3) (*a*) and (*d*) for a water-closet.

(*b*) If not situated wholly or partly within a building be constructed in accordance with the requirements contained in bye-law No. 2 (3) (*b*) and (*d*) for a water-closet.

(*c*) Be suitably paved throughout with hard, jointless and impervious material finished with a smooth surface not less than three inches above the surface of the adjoining ground and with a sufficient fall to the door of such earth-closet.

(*d*) Be provided with a space beneath the seat prescribed in paragraph (5) (*a*) of this bye-law constructed :—

(i) Of sufficient dimensions to admit of the receptacle prescribed in paragraph (5) (*b*) of this bye-law being so placed and fitted as effectually to prevent the deposit of any filth elsewhere than in such receptacle and to allow of the effectual application of dry earth or other deodorising substance to any filth in such receptacle.

(ii) With adequate means of access for cleansing such space and for placing therein and removing therefrom the receptacle for filth,

(iii) With the surface of the containing walls of such space rendered with hard and impervious material finished with a smooth surface ; and any woodwork abutting upon

such space so coated or treated as to be impervious to moisture.

(*e*) Be so constructed that :—

(i) The contents of the receptacles for filth and dry earth or other deodorising substance prescribed in paragraph (5) of this bye-law shall be protected from [illegible] any waste water or liquid refuse from any premises.

(ii) No part of such earth-closet nor any receptacle therein shall communicate with any drain.

(4) *Lighting and ventilation.*—Such earth-closet shall be provided with a sufficient opening for light and ventilation as near to the top as practicable and communicating directly with the external air.

(5) *Provision of seat, receptacles for filth and dry earth, and apparatus.*—Such earth-closet shall be provided with :—

(*a*) A suitable hinged seat fitted with a urine guide.

(*b*) A movable receptacle of a capacity not exceeding two cubic feet, constructed of galvanized iron or other equally suitable material, for the reception of filth.

(*c*) A suitably constructed receptacle of adequate capacity so placed as to admit of ready access for depositing therein a supply of dry earth or other deodorising substance and fitted with suitable means or apparatus for the effectual application of a sufficient quantity of dry earth or other deodorising substance to any filth deposited in the receptacle for filth.

Privies. 5.—A builder constructing a privy in connection with a building shall comply with the following requirements :—

(1) *Situation.*—Such privy shall be situated :—

(*a*) Twenty feet at least from any public room, any school, any dwelling house, and any building in which any person may be or may be intended to be employed in any manufacture, trade or business.

(*b*) One hundred feet at least from any well, spring, or stream of water used or likely to be used by man for drinking or domestic purposes, and in such a position as will not render any such water liable to pollution.

(*c*) In such a position and manner as to afford ready means of access for the purpose of cleansing such privy and of removing filth therefrom without such filth being carried through any public room, or school, or dwelling house, or any building in which any person may be or may be intended to be employed in any manufacture, trade or business, or any building used for the storage of food or drink for man.

(2) *Form of construction.*—Such privy shall be :—

(*a*) Constructed of brick, concrete or other suitable material and covered with a properly constructed roof provided with suitable means for discharging rain-water therefrom.

(*b*) Provided with a space beneath the seat prescribed in paragraph (4) (*a*) of this bye-law constructed :—

(i) Of sufficient dimensions to admit of the receptacle for filth prescribed in paragraph (4) (*b*) of this bye-law being

so placed and fitted as effectually to prevent the deposit of any filth elsewhere than in such receptacle.

(ii) With walls of brick, concrete, or other suitable material of adequate thickness rendered with hard and impervious material finished with a smooth surface; a riser of flagging, slate, rendered brickwork, or other suitable hard and impervious material finished with a smooth surface; and any woodwork abutting upon such space so coated or treated as to be impervious to moisture.

(iii) With a door in the back or one of the sides of such privy capable of being opened from the outside of such privy for the purpose of cleansing such place and removing therefrom or placing or fitting therein the receptacle for filth.

Provided always that in any case where the provision of such door is impracticable, the foregoing requirement shall be deemed to be complied with if the whole of such seat or a sufficient part thereof is so arranged and fitted as to afford adequate means for access and cleansing.

(*c*) Suitably paved with hard, jointless and impervious material finished with a smooth surface with the surface of the portion in front of the seat prescribed in paragraph (4) (*a*) of this bye-law not less than six inches and the space beneath such seat not less than three inches respectively above the surface of the adjoining ground and with suitable falls to the doors of such privy.

(*d*) Provided with a proper entrance door and fastenings.

(*e*) So constructed that:—

(i) The contents of the receptacle for filth shall be protected from any rainfall and any waste water or liquid refuse from any premises.

(ii) No part of such privy nor any receptacle therein shall communicate with any drain.

(3) *Lighting and ventilation.*—Such privy shall be provided with means of lighting and ventilation in accordance with the requirements contained in bye-law No. 4 (4) for an earth-closet.

(4) *Provision of seat and receptacle for filth.*—Such privy shall be provided with:—

(*a*) A suitable hinged seat fitted with a urine guide.

(*b*) A movable receptacle of a capacity not exceeding two cubic feet, constructed of galvanized iron or other equally suitable material, for the reception of filth.

Cesspool.

6.—A builder constructing a cesspool in connection with a building shall comply with the following requirements:—

(1) *Situation.*—Such cesspool shall be situated:—

(*a*) One hundred feet at least from any public room, any school, any dwelling house, any building in which any person may be or may be intended to be employed in any manufacture, trade or business, and any well, spring, or stream of water.

(*b*) In such a position as to afford ready means of access for the purpose of emptying and cleansing such cesspool and for

removing the contents therefrom without being carried through any public room, or school, or dwelling house, or any building in which any person may be or may be intended to be employed in any manufacture, trade or business, or any building used for the storage of food or drink for man.

(2) *Form of construction.*—Such cesspool shall be :—

(*a*) Constructed of concrete, or good brickwork bedded and grouted in cement and rendered on the inside surface with cement, with walls and floor at least nine inches in thickness, or otherwise constructed of suitable materials, and so as to be water-tight.

(*b*) Arched or otherwise properly covered over and provided with means of access furnished with a suitable air-tight cover.

(*c*) Provided with adequate means of ventilation.

(*d*) So constructed as not to communicate with any sewer.

Drainage Bye-laws.—Bye-laws made by the London County Council on March 6th, 1934, regulating the dimensions, form and mode of construction, and the keeping, cleansing and repairing of the pipes, drains and other means of communicating with sewers and the traps and apparatus connected therewith.

Interpretation of terms.

1.—In these bye-laws, unless the context otherwise requires :—

" Drainage work " means any pipe, drain, or other means of communicating with sewers and any trap or apparatus connected therewith.

" Subsoil drain " means a drain used or constructed to be used solely for conveying to any sewer (either directly or through another drain) any water that may percolate through the subsoil.

" Surface-water-drain " means a drain used or constructed to be used solely for conveying to any sewer (either directly or through another drain) any rain water from roofs or from ground surfaces whether paved or unpaved, but does not include a rain-water pipe.

" Rain-water pipe " means a pipe or drain situate wholly above ground and used or constructed to be used solely for carrying off rain water directly from roof surfaces.

" Sewage drain " means a drain used or constructed to be used for conveying solid or liquid waste matters to a sewer.

" Slop sink " means a sink constructed or adapted to be used for receiving solid or liquid excremental matter.

" Soil fitment " means a water-closet, slop sink, or a urinal.

" Waste-water fitment " means a bath, lavatory basin, bidet, or a sink other than a slop sink.

" Concrete " means a conglomerate of clean gravel, hard brick broken small, or other equally suitable aggregate, with a sufficient quantity of clean sand, well mixed with cement in the proportion of not less than one part by measure of cement to eight parts of other material.

" Cement " means good Portland or other suitable cement of at least equal quality.

" Sanitary authority " means the Council of the Metropolitan Borough in which any drainage work has been, is being or is about to be executed.

" Builder " means the builder, contractor or person actually executing any drainage work, and for the purpose of the bye-law No. 14 relating to the deposit of plans, sections, and particulars, means the builder, contractor or person intending actually to execute such work, but does not include a workman in the employ of such builder, contractor or person, and where, after written request by the sanitary authority, the owner or occupier of any premises on which any drainage work has been or is being executed fails to furnish such authority with the name and address of such builder, contractor or person, includes such owner or occupier.

Subsoil drains.

2.—(1) A subsoil drain shall not be connected directly with a sewage drain, or sewer, or a surface-water drain communicating with a sewage drain or sewer without the intervention of a suitable trap between such subsoil drain and such sewage drain, or sewer, or surface-water drain.

(2) A ventilating opening shall be provided to such subsoil drain as may be practicable to such trap, and shall communicate directly with the open air.

(3) Such trap and the drain between such trap and such sewage drain, or sewer, or surface-water drain, shall be constructed in the manner prescribed in bye-law No. 5 for sewage drains.

(4) The subsoil drain above such trap shall :—

(*a*) Be formed of earthenware field or other suitable pipes.

(*b*) Be properly laid to an adequate fall.

(*c*) Discharge into such trap.

Surface-water drains

3.—A surface-water drain shall be constructed in the manner prescribed in bye-law No. 5 for sewage drains.

Provided always :—

(*a*) That such drain may have an internal diameter of less than four inches.

(*b*) That where such drain is intercepted from a sewage drain or sewer by a suitable trap communicating directly with the open air and furnished with adequate means of access :—

(i) If the inlet to such drain is not less than ten feet distant from any building, the inlet may be in the form of an untrapped gully with adequate provision for catching sand or other detritus and covered with a grating, the bars of which shall be not more than three-eighths of an inch apart.

(ii) If such drain receives only rain water from roofs, the inlet to such drain may be in the form of a rain-water shoe, or an untrapped gully as prescribed in the foregoing paragraph (i).

Rain-water pipes.

4.—A rain-water pipe conveying to a sewer any rain-water shall discharge directly or by means of a channel into or over an inlet to :—

(*a*) A surface-water drain, or

(*b*) A sewage drain.

Provided always :—

(i) That where the inlet to such drain is in the form of a properly-trapped gully or other suitable trap such pipe or channel shall discharge above the level of the water in such gully or trap.

(ii) That a rain-water pipe shall not discharge into or connect with any soil pipe or soil ventilating pipe, or any waste pipe or waste ventilating pipe connected as described in bye-law No. 10 (2).

Sewage drains.

5.—(1) *Materials.*—A sewage drain shall be constructed of good sound pipes formed of cast-iron, glazed stoneware, or other equally suitable material.

(2) *Cast-iron pipes, etc.*—If the pipes, traps and fittings are constructed of cast-iron, they shall be effectually protected against corrosion by being coated on both the inside and outside with Dr. Angus Smith's solution, or other equally suitable solution, or by treatment in some other equally suitable manner, and the thickness of the pipes, traps and fittings, the weight of the pipes, the internal depth of the sockets, and the caulking space shall be in conformity with Table No. 1 in the schedule to these bye-laws.

(3) *Stoneware pipes, etc.*—If the pipes, traps and fittings are constructed of stoneware, they shall be of first quality properly glazed, and the thickness of the pipes, traps and fittings, the internal depth of the sockets, and the jointing space shall be in conformity with Table No. 2 in the schedule to these bye-laws.

(4) *Size, fall and line of drain.*—Every such drain shall :—

(*a*) Be of suitable size with a minimum internal diameter of four inches.

(*b*) Be laid with a suitable fall, and, where practicable, in a direct line.

(5) *Joints of drain.*—Every joint in such drain shall be made in the manner and with the jointing materials hereinafter prescribed or otherwise in an equally suitable and efficient manner and with equally suitable materials, and so as to preserve the continuity of the drain without obstruction, namely, if such drain be constructed of :—

(*a*) Cast-iron socketed pipes, the joints shall be made with a gasket of hemp or yarn and metallic lead properly caulked.

(*b*) Cast-iron flanged pipes, the joints shall be securely bolted together with some suitable insertion.

(*c*) Stoneware pipes, or pipes of material other than metal, such pipes shall be jointed with socket joints properly put together with a gasket of hemp or yarn and cement or other equally suitable materials.

(6) *Drain to be laid on concrete.*—Such drain shall be laid on a bed of concrete not less than six inches thick and projecting on each side of the drain to a width not less than six inches.

Provided always that if any such drain is constructed of cast-iron pipes above the ground and carried at least at each joint on adequate piers or other sufficient supports, the requirements of this paragraph shall not apply.

(7) *Concrete to be filled in.*—Concrete shall be filled in so that it shall extend to the full width of the concrete bed prescribed in the foregoing paragraph No. (6), and shall be haunched up to not less than half the external diameter of the pipe.

(8) *Junctions.*—Every branch drain shall join another drain obliquely in the direction of the flow of such drain and as near as practicable to the invert thereof.

(9) *Drains within or under buildings.*—(*a*) A sewage drain shall not be constructed so as to be within or under any building, except in any case where any other situation is impracticable.

(*b*) Where any such drain or part thereof is constructed within or under a building, such drain or such part thereof shall :—

(i) Be laid or fixed in a direct line where practicable, and be provided with adequate means of access.

(ii) If constructed of stoneware pipes, be laid on a bed of concrete as prescribed in the foregoing paragraph No. (6) and encased in concrete at least six inches thick.

(iii) If constructed of cast-iron pipes, be laid on a bed of concrete, and filled in and haunched up with concrete as prescribed in the foregoing paragraphs Nos. (6) and (7).

Provided always that if any such drain constructed of cast-iron pipes be above the ground and carried at least at each joint on adequate piers or other sufficient supports, requirement (iii) shall not apply.

(10) *Protection to drain beneath wall.*—Where any such drain is laid beneath a wall, it shall be protected at the part beneath the wall by means of a relieving arch, or other support, which shall not bear on the drain.

(11) *Inlets to drains within buildings.*—Such drain shall not be constructed in such a manner that there shall be within a building any inlet to such drain except such inlet as may be necessary from any soil fitment, or any waste-water fitment connected directly to such drain.

Provided always that a drain inlet other than those above-mentioned may be provided within a building if an external position is impracticable ; in which case such inlet shall be trapped by a suitable and efficient trap, as hereinafter prescribed in paragraph (12) of this bye-law, fitted with a suitable cover and provided where necessary with adequate means of ventilation to the external air.

(12) *Inlets to drains to be trapped.*—Every inlet, other than a ventilating pipe, to such drain shall be properly trapped by a suitable and efficient trap, and such trap shall be [illegible] and fixed so as to be capable of maintaining a water seal of :—

(*a*) Two inches where such inlet has an internal diameter of not less than three inches.

(*b*) Three inches where such inlet has an internal diameter of less than three inches.

(13) *Gratings to trapped gullies.*—Every trapped gully shall be covered with a grating, the bars of which shall be not more than three-eighths of an inch apart.

(14) *Trapping of drains from sewer.*—(*a*) If an intercepting trap * is provided to such drain, such trap shall be :—

(i) Suitable and efficient.

(ii) Provided with a raking or clearing arm fitted with a secure and suitable stopper as a means of access to the drain between such trap and the sewer.

(iii) Formed and fixed so as to be capable of maintaining a water seal of at least two inches.

(iv) Fixed at a point in such drain as near as may be practicable to the connection of such drain with the sewer.

(v) Provided with a manhole or other means of access sufficient for the purposes of clearing.

(*b*) If an intercepting trap is not provided to such drain any portion of such drain which may be within or under a building shall be constructed of cast-iron.

(15) *Drain to be water-tight.*—Such drain shall be constructed so as to be water-tight and to be capable of resisting a pressure of five feet head of water.

(16) *Means of access.*—Such drain shall be provided with adequate means of access, and every means of access shall be :—

(*a*) Constructed so as to be water-tight.

(*b*) Fitted with a suitable cover at the level of the adjoining ground surface.

Provided always :—

(i) That if such means of access is constructed within or under a building, or to a drain to which an intercepting trap has not been provided, it shall be furnished with a suitable screwed or bolted air-tight cover, and where the means of access is in the form of a manhole, such cover shall be fixed to the channel of the manhole and be in addition to the cover at the level of the adjoining ground surface prescribed in the foregoing paragraph (*b*).

(ii) That where the means of access is in the form of a manhole having a drain or channel fitted with an air-tight cover such manhole need not be water-tight.

(17) *Ventilation of drains.*—For the purpose of securing efficient ventilation of such drain the following requirements shall be complied with :—

(*a*) *Ventilating pipes.*—(i) If an intercepting trap is provided, at least two ventilating pipes shall be provided, one connected to the drain at a point as near as practicable to and on the inlet

* The Metropolis Management Act, 1855, and the London County Council (General Powers) Act, 1920, empower the sanitary authority to require the provision of an intercepting trap wherever they think fit.

side of the intercepting trap, and the other at a point as far distant as practicable from the intercepting trap.

(ii) If an intercepting trap is not provided, at least one ventilating pipe shall be provided, connected to the drain at a point as far distant as practicable from the sewer to which the drain is connected.

(iii) Every such pipe shall be carried up vertically to such a height and position as to prevent any nuisance or injury or danger to health arising from the emission of foul air from such pipe.

(*b*) *Size, means of access, construction, material and weight of ventilating pipes.*—Every such pipe shall :—

(i) Have an internal diameter of not less than three inches.

(ii) Be furnished at the foot thereof with a suitable air-tight access cap or cover.

(iii) Be otherwise constructed in the same manner and of the same material and weight as if such pipe were a soil pipe or a soil ventilating pipe.

(*c*) *Soil pipes, waste pipes and ventilating pipes used as drain ventilating pipes.*—The soil pipe or waste pipe or ventilating pipe of any soil fitment, or the waste pipe or ventilating pipe of any waste-water fitment where such pipe is connected directly to such drain, where the situation and diameter are in accordance with the requirements applicable to the pipe to be carried up from the drain, shall be deemed to provide the necessary means of ventilation.

(*d*) *Gratings to openings.*—The open end of every ventilating pipe or other pipe providing the necessary means of ventilation shall be fitted with a suitable grating or other cover having apertures of an aggregate area not less than the sectional area of the pipe for the purpose of preventing any obstruction in or injury to any pipe or drain connected therewith by the introduction of any substance through such open end.

Soil pipe and soil ventilating pipes.

6.—(1) *Materials, accessibility and protection.*—A soil pipe and soil ventilating pipe shall be :—

(*a*) Constructed of lead, copper, cast-iron, wrought-iron or other equally suitable material.

(*b*) Easily accessible throughout its course and adequately protected, where necessary, from damage.

(2) *Shape, size, fixing and outlet.*—Such pipe shall :—

(*a*) Be circular.

(*b*) Have an internal diameter of not less than three inches.

(*c*) Be securely fixed without unnecessary bends or angles.

(*d*) Be continued upwards without diminution of its diameter to such a height and position as to prevent any nuisance or injury or danger to health arising from the emission of foul air from such pipe.

(*e*) Have the open end fitted with a suitable grating or other cover as prescribed in bye-law No. 5 (17) (*d*) for drain ventilating pipes.

Provided always that in any case where the internal diameter of the outlet of the trap of any soil fitment connected to a soil pipe is more than three inches the internal diameter of such soil pipe shall not be less than the internal diameter of such outlet.

(3) *Coating, thickness, weight, etc.*—If such pipe be constructed :—

(*a*) Of lead, or copper, its weight shall be in conformity with Table No. 3 in the schedule to these bye-laws.

(*b*) Of cast-iron, it shall be protected against corrosion by being adequately galvanized, or coated or treated in the manner [illegible] bye-law No. 5 (2) for sewage drains, and its thickness, weight, the internal depth of the socket, and the caulking space shall be in conformity with Table No. 3 in the schedule to these bye-laws.

(*c*) Of wrought-iron, it shall be protected against corrosion by being adequately galvanized, or by treatment in some other equally suitable manner, and its thickness and weight shall be in conformity with Table No. 3 in the schedule to these bye-laws.

(4) *Joints.*—Every joint in such pipe shall be made in the manner and with the jointing materials hereinafter prescribed or otherwise in an equally suitable and efficient manner and with equally suitable materials, and so as to preserve the continuity of the pipe without obstruction, namely, if such pipe be constructed :—

(*a*) Of lead, the joints shall be of the kind known as burned, or plumber's wiped soldered joints.

(*b*) Of copper, the joints shall be of the kind known as compressed joints made with union nut or flanged couplings, or other equally suitable joints.

(*c*) Of cast-iron with sockets, the joints shall be (i) made with a gasket of hemp or yarn and metallic lead properly caulked, or (ii) screwed joints with galvanized shouldered cast-iron, wrought-iron or malleable-iron sockets.

(*d*) Of wrought-iron with sockets, the joints shall be screwed joints with galvanized shouldered cast-iron, wrought-iron or malleable-iron sockets.

(*e*) Of cast-iron with flanges, the joints shall be securely bolted together with some suitable insertion.

(*f*) Of wrought-iron with flanges, the joints shall be made with galvanized cast-iron, wrought-iron or malleable-iron flange unions or flanges securely bolted together with some suitable insertion.

(5) *Connection with rain-water, waste pipes and waste ventilating pipes.*—Such pipe shall not be connected with any rain-water pipe, or with any waste pipe or waste ventilating pipe unless such waste pipe or waste ventilating pipe is constructed of the materials and in the manner prescribed in bye-law No. 10 (2) relating to waste pipes and waste ventilating pipes.

(6) *No traps.*—There shall not be any trap in such pipe or between such pipe and any drain with which it is connected.

(7) *Pipe to be water-tight.*—Such pipe shall be constructed so as to be water-tight and to be capable of resisting a pressure of five feet head of water.

Connections of soil pipes, soil ventilating pipes, traps and drains.

7.—The connection of the trap of any soil fitment with a soil pipe, ventilating pipe or drain, or the connection of any soil pipe or ventilating pipe with a drain, shall be made in the manner and with the jointing materials hereinafter prescribed or otherwise in an equally suitable and efficient manner, and with equally suitable materials, and so as to preserve the continuity of the trap, pipe or drain without obstruction, namely :—

(*a*) The connection of a lead trap with a lead pipe shall be by a burned, or plumber's wiped soldered joint.

(*b*) The connection of a lead pipe or trap with a copper pipe or trap shall be by a plumber's wiped soldered joint.

(*c*) The connection of a lead pipe or trap with an iron pipe, trap or drain, shall be by means of a thimble or flanged ferrule of copper, brass or other suitable alloy connected with the lead pipe or trap by a plumber's wiped soldered joint, and with the iron pipe, trap or drain by a joint made with a gasket of hemp or yarn and metallic lead properly caulked, a screwed joint with a galvanized shouldered cast-iron, wrought-iron or malleable-iron socket, or galvanized cast-iron, wrought-iron or malleable-iron flange union or flanges securely bolted together with some suitable insertion.

(*d*) The connection of a lead pipe or trap with a stoneware pipe, trap or drain, shall be by means of a thimble or ferrule as described in (*c*) connected with the lead pipe or trap by a plumber's wiped soldered joint and with the stoneware pipe, trap or drain by a joint made with a gasket of hemp or yarn and cement.

(*e*) The connection of a copper trap with a copper pipe shall be by means of a union nut or flanged coupling.

(*f*) The connection of a copper pipe or trap with an iron pipe, trap or drain shall be by means of a thimble or flanged ferrule of copper, brass or other suitable alloy connected with the copper pipe or trap by a union nut or flanged coupling, and with the iron pipe, trap or drain by a joint made with a gasket of hemp or yarn and metallic lead properly caulked, a screwed joint with a galvanized shouldered cast-iron, wrought-iron or malleable-iron socket, or galvanized cast-iron, wrought-iron or malleable-iron flange union or flanges securely bolted together with some suitable insertion.

(*g*) The connection of a copper pipe or trap with a stoneware pipe, trap or drain shall be by means of a thimble or ferrule as described in (*f*) connected with the copper pipe or trap by a union nut or flanged coupling, and with the stoneware pipe, trap or drain by a joint made with a gasket of hemp or yarn and cement.

(*h*) The connection of an iron pipe or drain with an iron trap shall be by a joint made with a gasket of hemp or yarn and metallic lead properly caulked, a screwed joint with a galvanized shouldered cast-iron, wrought-iron or malleable-iron socket, or galvanized cast-iron, wrought-iron or malleable-iron flange union or flanges securely bolted together with some suitable insertion.

(*i*) The connection of an iron pipe, trap or drain with a stoneware pipe, trap or drain and the connection of a stoneware trap with a stoneware pipe or drain shall be by a joint made with a gasket of hemp or yarn and cement.

Slop sinks and urinals.

8.—(1) *Materials, form of construction and flushing.*—A slop sink shall be constructed :—

(*a*) Of glazed earthenware, enamelled fireclay, or other equally suitable material.

(*b*) Of such shape, and furnished with such flushing rim, water supply, and apparatus as to provide for the effectual flushing and clearing of the slop sink and the trap and waste pipe connected therewith.

(2) *Materials, form of construction and size of waste pipes and waste ventilating pipes.*—A waste pipe from any slop sink, or any urinal, and a waste ventilating pipe shall be constructed of the materials and in the manner prescribed in bye-laws Nos. 6 and 7 for soil and soil ventilating pipes, and Table No. 3 in the schedule to these bye-laws.

Provided always that the internal diameter of the waste pipe of any urinal may be not less than two inches in the case of a urinal having not more than two basins, or two stalls or compartments not exceeding four feet six inches in total width, or one and a half inches in the case of a single urinal basin.

(3) *Trapping of waste pipes.*—Every such waste pipe shall be trapped immediately beneath such slop sink or urinal by a suitable and efficient tubular trap or other equally suitable and efficient trap and such trap shall :—

(*a*) Be constructed of lead, copper, cast-iron coated or treated as a protection against corrosion in the manner provided for in bye-law No. 5 (2) for sewage drains, galvanized wrought-iron, galvanized malleable-iron, glazed earthenware, or other equally suitable material.

(*b*) Have an outlet with an internal diameter not exceeding the internal diameter of the waste pipe to which it is connected.

(*c*) Be provided with adequate means for inspection and clearing.

Provided always that where two or more urinal basins or stalls are fixed in a range the waste pipes may discharge without the interposition of a trap into a semi-circular and accessible open channel of glazed stoneware or other equally suitable material formed or fixed in or on the floor immediately beneath or in front of such basins or stalls, but not extending laterally beyond such range, and discharging into a suitable and efficient trap constructed and fixed in accordance with this bye-law, and as prescribed in bye-law No. 5 (11) and (12) for sewage drains.

Ventilation of traps.

9.—(1) If the soil pipe or waste pipe of any soil fitment shall be in connection with any other such fitment, or if such soil pipe or waste pipe shall be in connection with the waste pipe of any waste-

water fitment, the trap of every such soil fitment or waste-water fitment shall be ventilated in the following manner :—

A trap ventilating pipe shall—

(*a*) Be connected with the trap or the branch soil pipe or waste pipe :—

(i) At a point not less than three nor more than twelve inches from the highest part of the trap.

(ii) On that side of the water seal which is nearest to the soil pipe or waste pipe.

(iii) In the direction of the flow.

(*b*) Be carried into the open air to a point as high as the top of the soil ventilating pipe or waste ventilating pipe and have the open end fitted with a suitable grating or other cover constructed in the manner prescribed in bye-law No. 5 (17) (*d*) for gratings to drain ventilating pipes, or into the soil ventilating pipe or waste ventilating pipe at a point above the highest fitment connected with such soil pipe or waste pipe.

(*c*) Where the vertical distance between the invert of the outlet of the lowest trap connected with the soil pipe or waste pipe and the invert of any horizontal pipe or drain into which such soil pipe or waste pipe discharges or is connected is less than ten feet, be continued downwards and connected with (i) the soil pipe, waste pipe or drain at a point not less than nine inches and not more than two feet below the invert of the lowest branch soil pipe or waste pipe connection, with adequate means of inspection at the point of connection, or (ii) a manhole in the line of such drain.

(2) The branch and main trap ventilating pipes respectively shall have in all parts an internal diameter of not less than :—

(*a*) Two inches where connected with a soil pipe, or a waste pipe three inches or more in internal diameter.

(*b*) Two-thirds of the respective internal diameters of the branch and main waste pipes where the internal diameters of such pipes are less than three inches, with a minimum internal diameter of one and a quarter inches.

(3) Every such trap ventilating pipe shall be constructed of the materials and in the manner prescribed in bye-laws Nos. 6 and 7 and Table No. 3 in the schedule to these bye-laws relating to soil pipes and soil ventilating pipes.

Provided always that if the internal diameter of such pipe is less than one and a half inches such pipe shall be constructed of a suitable non-ferrous material.

Waste-water fitments.

10.—A waste pipe from a waste-water fitment, a waste ventilating pipe, a trap ventilating pipe, and any trap connected therewith, shall have an internal diameter of not less than one and a quarter inches and shall be constructed of the materials and in the manner hereinafter prescribed in paragraphs (1) or (2) of this bye-law.

(1) If such waste pipe is constructed so as to discharge over or into a properly trapped gully, such waste pipe and any ventilating

pipe and trap connected therewith shall be in conformity with the following requirements :—

(*a*) *Materials and fixing.*—Such waste pipe, ventilating pipe and trap shall be :—

(i) Constructed of lead, copper, cast-iron, wrought-iron, glazed stoneware, or other equally suitable material.

Provided always that if the internal diameter of such pipe or trap is less than one and a half inches such pipe or trap shall be constructed of suitable non-ferrous material.

(ii) Securely fixed.

(*b*) *Coating, thickness, weight, joints, connections, etc.*—If such pipe or trap be constructed :—

(i) Of lead, copper, cast-iron, or wrought-iron, it shall be in conformity with the requirements prescribed in bye-laws Nos. 6 (3), (4) and 7 for soil pipes and soil ventilating pipes and Tables No. 3 in the schedule to these bye-laws.

(ii) Of stoneware, it shall be in conformity with the requirements prescribed in bye-laws Nos. 5 (3), (5) (*c*) and 7 for stoneware drains.

Provided always that the joints and connections may be made in an equally suitable and efficient manner.

(*c*) *Trapping of waste pipes.*—Every such waste pipe shall be trapped immediately beneath such fitment by a suitable and efficient tubular trap or other equally suitable and efficient trap, and such trap shall :—

(i) Be constructed in the manner prescribed in bye-law No. 8 (3) (*b*) and (*c*) for slop sinks and urinals.

(ii) Be formed and fixed so as to be capable of maintaining a water seal of at least one and a half inches.

Provided always :—

(i) That where two or more baths or lavatory basins are fixed in a range the waste pipes may discharge without the interposition of a trap into a semi-circular and accessible open channel of glazed stoneware or other equally suitable material formed or fixed in, on or above the floor immediately beneath such baths or lavatory basins and discharging over or into a suitable and efficient trap constructed and fixed as prescribed in this bye-law or as prescribed in bye-law No. 5 (12) for sewage drains.

(ii) That the waste pipe of any sink fixed in an outbuilding not approached directly from an occupied building need not be trapped if such waste pipe does not exceed three feet six inches in length and discharges over or into a suitable and efficient trap as prescribed in bye-law No. 5 (12) for sewage drains.

(*d*) *Ventilation of waste pipes.*—Where any such waste pipe is connected with two or more such fitments fixed on different storeys of a building it shall be constructed in the manner prescribed in bye-law No. 6 (2) (*d*) and (*e*) for soil pipes and soil ventilating pipes.

(*e*) *Ventilation of traps.*—In order to preserve the seal of the trap of any such fitment such trap shall be ventilated whenever necessary by a ventilating pipe carried to such a position as to prevent any nuisance or injury or danger to health arising from the emission of foul air from such pipe; and where such pipe is connected to the traps of two or more such fitments fixed on different storeys of a building it shall be carried up as high as the top of the waste ventilating pipe and have the open end fitted with a suitable grating or other cover constructed in the manner prescribed in bye-law No. 5 (17) (*d*) for gratings to drain ventilating pipes, or into the waste ventilating pipe at a point above the highest fitment.

Every such trap ventilating pipe shall be connected with the trap or the branch waste pipe :—

(i) At a point not less than three nor more than twelve inches from the highest part of the trap.

(ii) On that side of the water seal which is nearest to the waste pipe.

(iii) In the direction of the flow.

The branch and main trap ventilating pipes respectively shall have in all parts an internal diameter of not less than two-thirds of the respective internal diameters of the branch and main waste pipes.

Provided always that where the internal diameter of the waste pipe exceeds three inches the internal diameter of such ventilating pipe need not be greater than two inches.

(*f*) *Waste pipes to discharge in the open air over or into a trapped gully.*—Every such waste pipe shall be taken through an external wall of the building and shall discharge in the open air over a properly trapped gully or into such gully above the level of the water therein.

Provided always that where it is impracticable to discharge such pipe in the open air it may discharge within the building into a trap constructed in the manner prescribed in bye-law No. 5 (11) and (12) for sewage drains and above the level of the water in such trap.

Such waste pipe shall not discharge into or connect with any :—

(i) Hopper head.

(ii) Gutter provided or used for the conveyance of rain water.

(iii) Pipe constructed or intended to be used for conveying rain-water unless such pipe is in its entirety in conformity as regards the manner of construction, material and weight with the requirements in paragraph (1) of this bye-law, and unless the rain-water inlet or inlets to such pipe are so situated and constructed as to prevent any nuisance or injury or danger to health arising from the emission of foul air from such pipe.

(2) If such waste pipe or ventilating pipe is connected directly with any sewage drain, or sewage drain ventilating pipe, or the soil

or waste pipe or ventilating pipe of any soil fitment, such waste pipe, ventilating pipe, and any trap connected therewith shall be in conformity with the following requirements :—

(*a*) *Materials, form of construction, and size.*—Such waste pipe and ventilating pipe shall be constructed of the materials and in the manner prescribed in bye-laws Nos. 6 and 7 for soil pipes and soil ventilating pipes and Table No. 3 in the schedule to these bye-laws.

Provided always that the internal diameter of such waste pipe or ventilating pipe may be less than three inches, but shall be not less than one and a quarter inches.

(*b*) *Trapping of waste pipes.*—Every such waste pipe shall be trapped immediately beneath such fitment by a suitable and efficient tubular trap or other equally suitable and efficient trap, and such trap shall :—

(i) Be constructed in the manner prescribed in bye-laws Nos. 5 (12) for sewage drains, and 8 (3) (*a*), (*b*) and (*c*) for slop sinks and urinals.

(ii) Be ventilated in the manner prescribed in bye-law No. 9 for the ventilation of traps of soil fitments.

Provided always that if the internal diameter of such pipe or trap is less than one and a half inches such pipe or trap shall be constructed of suitable non-ferrous material.

Marking of pipes, traps, etc.

11.—No pipe, trap, apparatus or fitment bearing or marked with the letters or words " L.C.C." or " London County Council " or any reference thereto shall be used in connection with any drainage work unless such pipe, trap, apparatus or fitment is in conformity with the requirements of these bye-laws.

Maintenance in state of repair, etc.

12.—The owner of any house or building shall at all times keep and maintain in a proper state of repair and in proper working order all drainage work in or in connection with such house or building, and he shall at all times keep and maintain in conformity with these bye-laws all such drainage work constructed in accordance with these bye-laws.

Alteration, reconstruction and repair.

13.—No alteration, partial or entire reconstruction, or repair of any drainage work constructed in accordance with these bye-laws shall be made so that by reason of such alteration, partial or entire reconstruction or repair any such drainage work will not be in conformity with these bye-laws.

The alteration, partial or entire reconstruction, or repair of any drainage work shall, so far as practicable, be carried out so as to comply with these bye-laws in respect of such alteration, partial or entire reconstruction or repair.

Plans and notices of drainage work.

14.—(1)—(*a*) *Deposit of plans, etc., of drainage work.*—Every person about to construct, reconstruct, or alter any drainage work, shall deposit or cause to be deposited in duplicate with the sanitary authority at their office such plans, sections and block plan, clearly and indelibly made on cloth or linen, and such detailed description

and particulars of the proposed construction, reconstruction or alteration as may be necessary for the purpose of enabling such authority to ascertain whether such construction, reconstruction or alteration will be in accordance with the statutory provisions relative thereto and with these bye-laws.

In the case of any addition to or alteration of any drainage work so much of the existing work shall also be shown on such plans and sections as will enable the sanitary authority to see the relative positions of the new and old work, and if plans and sections of the existing work have previously been deposited the builder or person about to carry out the new work shall furnish or cause to be furnished to the sanitary authority the date of the previous deposit.

The plans, sections, detailed description and particulars hereinbefore mentioned shall be signed by or on behalf of such person and deposited seven days at least before such construction, reconstruction or alteration is commenced, and in the case where such construction is in connection with a building to be erected seven days at least before commencing the erection of the building.

(*b*) *Plans, sections and particulars.*—Such plans and sections shall be drawn to a scale (except in the case of block plans) of not less than one inch to every sixteen feet, and shall show :—

(i) The position or every soil fitment, waste-water fitment, apparatus and trap in connection therewith.

(ii) The fall of every drain.

(iii) The position and size of every drain, means of access, trap, gully, soil pipe, waste pipe, ventilating pipe and rain-water pipe.

(iv) The height and position of every chimney belonging to and the position of every window or other opening into the building in connection with which such work is to be executed within a distance of twenty feet from the open end of a soil pipe or ventilating pipe.

(v) The levels of the lowest floor of the building in connection with which such work is to be executed and the adjoining street.

(vi) The level of any yard, area, ground or open space in connection with such building.

(vii) The scale to which such plans are drawn.

(*c*) *Block plan.*—Such block plan shall be drawn to a scale of not less than one inch to every eighty-eight feet and shall show :—

(i) The premises upon which such work is to be carried out.

(ii) The position of the buildings on such premises, and so much of the properties adjoining thereto as may be affected by such work.

(iii) The names of the streets or thoroughfares immediately adjoining such premises, and the number or designation of such premises.

(iv) The lines, size, depth and inclination of the proposed drains, and, so far as can be ascertained without opening the ground, the lines, size, depth and inclination of the existing drains, and the arrangements for the ventilation of the drains

SCHEDULE *

TABLE No. 1.—*Cast-iron Drain Pipes.*

Internal diameter.	Internal depth of socket not less than	Caulking space not less than	Thickness of metal for pipes, traps and fittings not less than	Weight of pipes (including socket and beaded spigot or flanges) not less than
Inches.	Inches.	Inches.	Inches.	
2	2¾	¼	¼	42 lbs. per 6-ft. length.
3	3	5/16	5/16	98 ,, 9-ft. ,,
4	3	5/16	⅜	157 ,, ,, ,,
5	3	⅜	⅜	186 ,, ,, ,,
6	3½	⅜	⅜	225 ,, ,, ,,
7	4	⅜	7/16	316 ,, ,, ,,
8	4	⅜	15/32	370 ,, ,, ,,
9	4	⅜	½	441 ,, ,, ,,

TABLE No. 2.—*Stoneware Drain Pipes.*

Internal diameter.	Internal depth of socket not less than	Jointing space not less than	Thickness of pipes, traps and fittings not less than
Inches.	Inches.	Inches.	Inches.
3	2	5/16	7/16
4	2	⅜	½
5	2¼	7/16	9/16
6	2¼	7/16	⅝
7	2¼	7/16	11/16
8	2½	½	11/16
9	2½	½	¾

TABLE No. 3.—*Soil Pipes, Waste Pipes and Ventilating Pipes.*

	Lead.	Copper.	Cast-iron.				Wrought-iron.	
Internal diameter.	Weight per yard not less than	Weight per yard not less than	Internal depth of socket not less than	Caulking space not less than	Thickness of metal for pipes, traps and fittings not less than	Weight of pipes per 6-ft. length (including socket and beaded spigot or flanges) not less than	Thickness of metal for wrought-iron pipes and wrought-iron, cast-iron or malleable-iron traps and fittings not less than	Weight of pipes per yard (not including sockets or flanges) not less than
Inches.	lbs.	lbs.	Inches.	Inches.	Inches.	lbs.	Inches.	lbs.
1¼	6·25	2·25	—	—	—	—	0·176†‡	7·94‡
1½	7·5	2·7	2¼	¼	3/16	22	0·192†	9·82
2	10·0	4·17	2½	¼	3/16	24	0·192†	12·51
2½	12·5	5·19	2¾	¼	3/16	30	0·212†	17·64
3	15·0	7·11	2¾	¼	3/16	35	0·212†	20·82
3½	19·5	9·33	3	¼	3/16	41	0·212†	23·97
4	22·5	11·85	3	¼	3/16	46	0·212	27·13
4½	29·0	13·29	3¼	5/16	¼	66	0·212	30·3
5	41·0	14·76	3¼	5/16	¼	78	0·212	33·45
6	57·0	17·64	3½	5/16	¼	92	0·212	39·78

* It should be noted that the first column in each of the three tables in the schedule is an index to the particulars given in the table and does not fix the internal diameter of any pipe.

The weights of pipes given in the tables are minimum weights and will not be suitable in all circumstances. In determining the weight to be used, regard must be had to (*a*) the strength of pipe needed to resist the internal working "head" or pressure, and (*b*) the possibility of such pipe being used for the conveyance of hot liquids.

† See terms of allowance of the bye-laws by the Minister of Health.

‡ In virtue of bye-laws 6, 8, 9 and 10, the practical application of this size of pipe in wrought-iron is limited to partial reconstruction or air of existing pipes of less than one and a half inches internal diameter.

—the existing drains and the proposed drains to be distinctively indicated by different colours.

(v) The points of the compass.

Provided, nevertheless, that it shall not be necessary to deposit a block plan in any case where the plans, sections, and particulars deposited in accordance with paragraph (*b*) of this bye-law clearly show the particulars required to be shown on a block plan.

(*d*) *Detailed description.*—Such detailed description shall sufficiently describe the intended mode of constructing such soil fitment, waste-water fitment, apparatus, trap, drain, means of access, gully or pipe.

(2).—*Notice of drainage work.*—Such person shall also serve or cause to be served upon the sanitary authority at their office at least twenty-four hours' notice in writing of the day and time at which any work of construction, partial or entire reconstruction, or alteration is to be commenced.

(3) *Urgent cases.*—In any case in which any partial or entire reconstruction or alteration of drainage work must be carried out at once, the builder may, in lieu of depositing the plans, sections detailed description and particulars and serving the notice referred to in this bye-law before commencing such work, forthwith send to the sanitary authority a notice in writing of such work.

Provided always that he shall within fifteen days of the commencement of such work make or cause to be made the deposits required by this bye-law.

(4) *Exemption.*—Nothing in this bye-law shall require the deposit of any plan or section in the case of any repair which does not involve the alteration or the entire reconstruction of any drainage work.

Penalty.

15.—Every builder who shall offend against any of the foregoing bye-laws and every owner who shall offend against bye-law No. 12, and every person who shall offend against bye-law No. 14, shall be liable for every such offence to a penalty of two pounds, and in the case of a continuing offence to a further penalty of twenty shillings for each day after written notice of the offence is given in accordance with Section 202 of the Metropolis Management Act, 1855.

Provided always that no proceedings shall be taken against an owner for an offence against bye-law No. 12, unless and until written notice has been served upon him by the sanitary authority requiring him within a period as is specified in the notice to comply with the bye-law and he has failed to comply with the bye-law within the time so specified.

Bye-laws not to apply to City.

16.—These bye-laws shall not extend to the City of London.

Repeal.

17.—From and after the date of the approval of these bye-laws the bye-laws made by the London County Council under section 202 of the Metropolis Management Act, 1855, and the Metropolis Management Acts Amendment (Bye-laws) Act, 1899, and approved by the Minister of Health on 4th March, 1930, shall be repealed, except as regards any drainage work commenced before the date of

the confirmation of these bye-laws or any drainage work not so commenced, but of which plans shall either have been approved by the sanitary authority before such date, or have been sent to the sanitary authority one month at least before such date, and shall not have been disapproved by the sanitary authority. Provided that this exception shall not be deemed to prohibit any such work from being executed in accordance with, or so as not to contravene, the foregoing bye-laws.

The foregoing bye-laws were made by the Council on the sixth day of March, 1934, and confirmed by the Council on the fourteenth day of March, 1934.

The seal of the Council was hereunto affixed on the second day of July, 1934.

(L.S.) G. H. GATER,
Clerk of the London County Council.

THE COUNTY HALL, S.E.1.

The foregoing bye-laws except so much of the table numbered 3 (three) in the schedule as required a greater thickness of metal than is specified in the second column in this certificate for malleable-iron traps and fittings of an internal diameter not less than is specified in the first column opposite to each thickness—

inches.	inches.
1¼	·13
1½	·14
2	·16
2½	·17
3	·19
3½	·19

are hereby allowed by the Minister of Health this twenty-eighth day of July, 1934.

(L.S.) E. H. RHODES,
Assistant Secretary, Ministry of Health.

Sewage Disposal.—The following are some of the methods of disposing of excreta and sewage :—

1. *Conservancy.*—In communities which are scattered as in ordinary village conditions it is possible to adopt a treatment which conserves the manurial value of the fæcal matter. The excreta is collected from the pail or earth closets daily and dug into the ground.

In more densely populated areas, one of the following water carried systems will usually have to be adopted.

2. *Water Carried Systems.—Discharge into Sea.*—The discharge of sewage into streams or in the sea is governed by the regulations of the Rivers Pollution Prevention Acts, 1876–93; these prohibit the discharge of sewage into non-tidal streams or rivers without purification and rendering innocuous. Sewage may by consent of the Minister of Health be discharged into tidal rivers or into the sea. Sewage should only be discharged into tidal rivers when the outlet is near the sea and there is no possibility of its being returned with the flood tide. Where the discharge is into the sea the outfall should be below low water mark. It is preferable that the discharge should only be made during the ebb tide, provision being made for storage during the period of flood tides.

3. *Irrigation* consists in passing the sewage over land, of which loamy porous soil is the best suited; clay is unsuitable owing to the liability to cracks which would allow the sewage to pass through unpurified. Where the land is open and pervious the more solid parts of the sewage, as well as the finely suspended organic matters, admit of being liquefied in the interstices of the soil.

The land may be used for growing cereal or root crops, or for grazing, according to the amount of liquid which is discharged on to it.

4. *Filtration (Intermittent Downward).*—In this system the surface of suitable light land is laid out in level beds and furrowed. The sewage is then led on by carriers and distributed by the furrows, passing downwards through the soil from which it escapes in a more or less purified condition by means of subsoil drains into some watercourse or stream.

The preliminary straining of the coarser solids is advisable.

Land used for sewage filtration requires constant aeration by being dug over.

5. *Precipitation.*—The object of precipitation is, by chemical treatment of the sewage, to form chemical compounds which by settling will draw down the suspended polluting matters. Precipitation is generally adopted as a preliminary treatment to irrigation or filtration.

Many precipitants have been introduced, such as lime; herring brine and lime; lime and sulphate of iron; lime and alumina; alum, blood, clay and charcoal; ferric aluminate; and patented compounds such as ferozone, aluminoferric, oxynite, etc.

The sludge is carried away and deposited in the sea, as in the case of the London sewage works, or is compressed by hydraulic machinery into a form convenient for transport to be used on the land.

In lieu of natural land filters, precipitation is in some systems followed by filtration by artificial filters such as in the case of the "International" process where, after precipitation by "Ferozone," the effluent is filtered through polarite filters, the oxygen contained in the pores of the polarite destroying the organic matter in solution.

6. *Electrolysis.*—The sewage is passed through channels in which cast-iron plates are set, and connected to the terminals of a dynamo. The process results in a great reduction of the organic matter in solution, which is oxidised by the free oxygen evolved.

7. *Bacteriolysis.* — Sewage always contains micro-organisms, which have a solvent action on the organic matter which it contains. These organisms may be either:

(*a*) Anaerobic, *i.e.*, those which thrive without air, and have a septic action upon the organic matter in the sewage, liquefying about 25 to 30 per cent. of the suspended solids, but having no purifying action, or

(*b*) Aerobic, *i.e.*, those which require a free supply of air, and purify the sewage.

In the septic tank system and in the sewers, the sewage is subjected to the action of anaerobic bacteria which break up the sewage.

When sewage is applied to suitably constructed filters, which permit a plentiful supply of oxygen to the body of the

filters, the growth of aerobic bacteria is facilitated and purification is effected.

Septic Tank System.—A tank of ample size is provided from which light and air is excluded. The sewage enters and escapes by submerged inlet and outlet respectively. The organisms present are under these conditions greatly [illegible], and decomposition occurs, of which the ultimate products are water, ammonia, carbonic acid and other gases. The effluent from the septic tank is discharged on to filters, where it is purified by aerobic action.

In the septic tank a scum forms which does not, however, exceed a depth of some 3 inches ; grit, etc., is deposited on the bottom of the tank. No chemicals are required, no sludge is formed and the ultimate filtrate is of a high degree of purification.

This process is the best-known method of dealing with the sewage from isolated country houses.

Filtration by bacterial filters, often referred to as the Dibdin or Sutton System from the name of its originator or the venue of his experiments, consists in the treatment of the sewage (after preliminary screening or settlement to remove the grosser solids) by a series of bacteria beds. These are formed with coke breeze, burnt ballast, or other suitable material ; half the beds are " coarse grain beds " having filtering material exceeding ½-inch gauge, the other half are " fine grain beds," material under ½-inch gauge, but excluding dust. The sewage is run successively into a coarse grain bed and a fine grain bed.

The beds are worked on a " cycle," allowing one hour for filling, two hours to remain full, one hour for emptying, and four hours to rest empty. At intervals of six or eight days each bed is allowed to remain empty for 24 hours.

Many materials varying from chalk to slate have been found suitable, but as a rule porous materials effect greater purification than impervious material, due apparently to the increased contact area of the former.

Cesspools are provided for the disposal of the sewage of country residences and mansions or districts that are not sewered. The drain is continued from the houses for a

distance of 100 feet or more to convey the sewage which is passed through a disconnecting trap into the cesspool. The cesspool should be not less than 100 feet from any well, spring, or stream of water.

The cesspool is made sufficiently large to contain a quantity of sewage equal to from a week to a 3 months' accumulation as may be desired. The sewage is pumped out of the cesspool into specially constructed tanks mounted on a chassis, to which a petrol pump is attached, a flexible hose conveys the contents to the tank with the least possible annoyance. The cesspool is built with impervious sides and bottom, with good brickwork bedded and grouted in cement, properly rendered inside with cement, and with an outside backing of at least 9 inches of well-puddled clay around and beneath such brickwork, to make the cesspool watertight. Figs. 827 and 828 illustrate this arrangement.

The cesspool should be ventilated. A cesspool can be arranged as a septic tank, and have an overflow pipe (with the end submerged below the scum on the surface of the sewage) to empty into a filter bed, or into agricultural land by a system of sub-irrigation pipes.

For the provisions for cesspools see the Bye-laws of the London County Council with respect to cesspools (see page 935).

Sewerage.—The various systems of sewerage have been referred to (p. 953). For the conveyance of sewage, stoneware pipes similar to those used for house drains may be used provided the diameter does not exceed 2 feet. For larger sewers, brickwork or concrete is usually employed.

Circular sewers may be made in brickwork, in two or more rings according to the diameter of the sewers. A concrete foundation is first laid, the upper surface of which is formed to receive the brick invert; when the invert and the brickwork up to the level of the centre has been laid, centering is provided on which the brickwork is completed. Encasement of concrete is provided according to the needs of the particular case. Blue Staffordshire or similar bricks should be used for the invert, but are not necessary for the upper part.

Fig. 827.

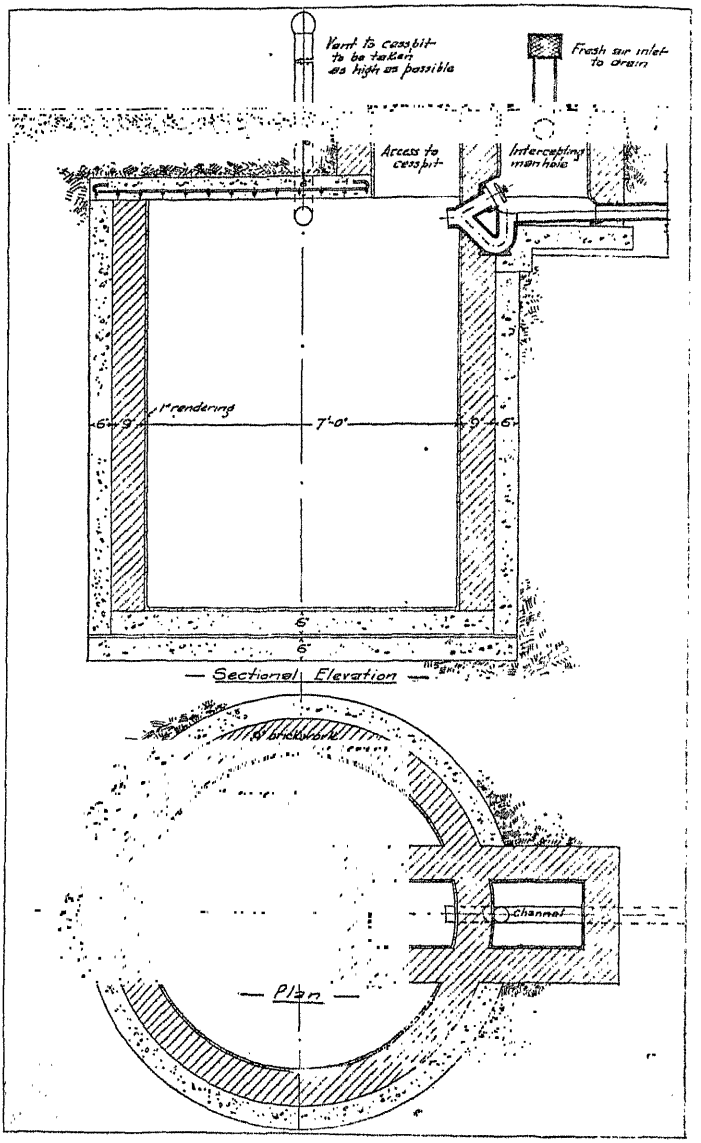


Fig. 828.

Figs. 827—828.

Egg-shaped Sewers.—These, as shown in Fig. 829, are employed to increase the velocity of the flow by making the current deeper ; the advantage of this shape over the circular one is most apparent when there is only a small quantity of water in the drain. The method of striking the curves is shown in Fig 831. The construction is as follows : a bed of concrete, with a horizontal surface levelled to the proper falls, is laid in the trench ; on this the invert is bedded, this being made of blue Staffordshire bricks or vitrified stoneware of the shape shown. Concrete is then shot in behind a centre to the height of the brick invert ; the bricks for the latter are now laid generally in two half-brick rings, the upper part is then completed on a centre, and the earth filled in.

Sewers are usually built of ordinary shaped bricks, which leave a large wedge-shaped joint ; to avoid this defect specially shaped bricks are now largely employed.

Any connections that are made to the sewer from the house drains should be taken through the sides just above the spring of the covering arch, which is the highest point to which water would rise under ordinary conditions. All house drains should have an iron flap trap on the outlet end to prevent any back flow, should the sewer become filled or the water line rise above the normal, as it often does after heavy rains ; these flap traps also help to prevent vermin entering the house drains from the sewer.

Sewers should be provided with manholes at distances not exceeding 200 feet, between which the sewer should lie in a straight line both in plan and gradient.

Figs. 832 to 835 show three views illustrating sections of sewer (side inlet) manhole, and ventilating shaft in the centre of the roadway.

Concrete Drains.—Of late years concrete has been largely used for the manufacture of drain pipes 6 inches in diameter and upwards, and in the construction of sewers up to 7 feet in diameter. They are made in concrete only and of ferro-concrete. It is essential in concrete pipes that a non-porous concrete be used ; the method for determining the proportions of the components should be as described in

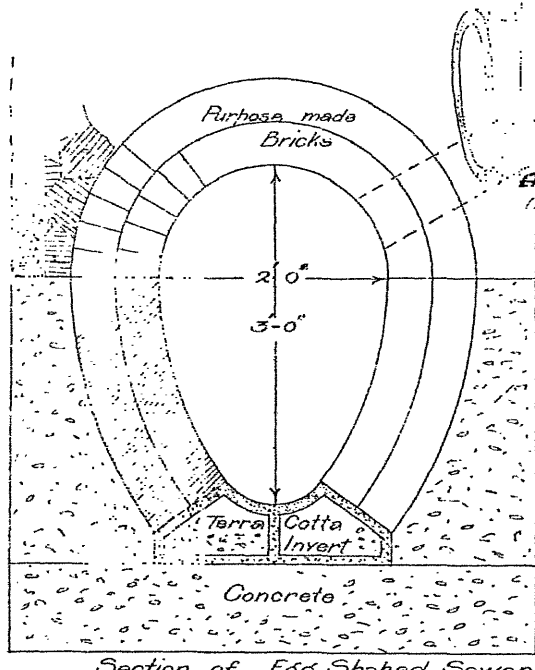


Fig. 829.

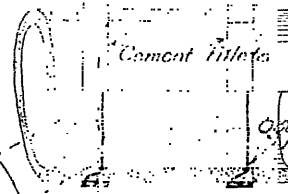


Fig. 830.

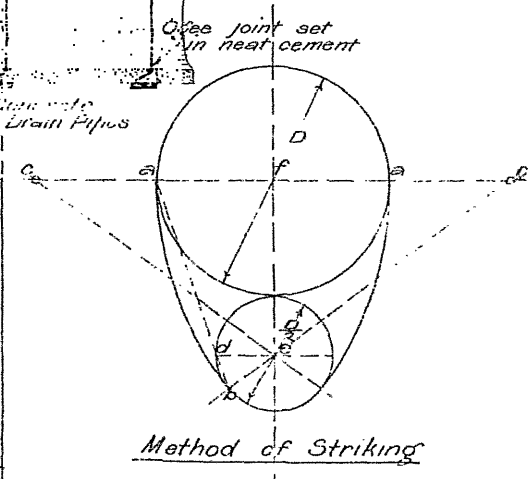


Fig. 831.

Figs. 829—831.

Fig. 832.

Figs. 832—835.

the chapter on ferro-concrete. The pipes are cast, varying from 2 feet in length and from 1½ to 3 inches in thickness, in wood or metal moulds consisting of a core and outer casing; the concrete is added in small quantities and well

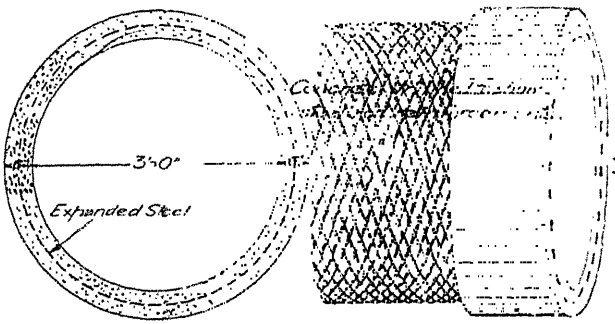


Fig. 836.

Concrete Sewer with Expanded Metal Reinforcement

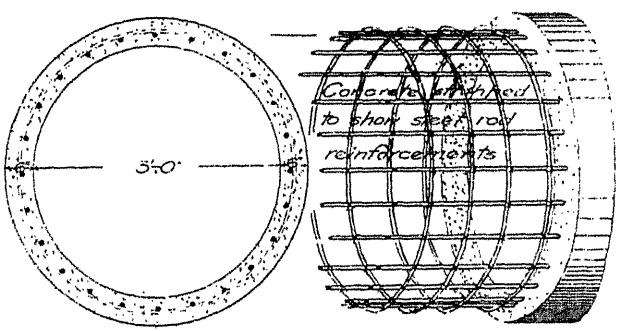


Fig. 837.

Concrete Sewer with Steel Rod Reinforcements

Figs. 836—837.

tamped to make the mass as close and dense as possible; the joints are usually of the ogee form. To harden and to ensure the non-porosity of the pipes, they may be dipped in a bath of silicate of soda.

Reinforced Concrete Drains.—Drains, sewers, and water conduits are now being largely made in ferro-concrete. They may be moulded *in situ*, forming monolithic blocks

or made in sections and jointed. Where great pressures are anticipated the former method is employed. The reinforcements usually consist of longitudinal bars laced to circular or other shapes, or of expanded metal, to form a network of steel, as shown in Figs 836 and 837.

The internal surfaces of sewers moulded *in situ* are often floated in Portland cement to ensure smooth and non-porous surfaces.

Hydraulic Formulæ—Fall for Drains.—The drains should be laid with a fall sufficient to give a velocity to effectually scour away all heavy matters and yet not too rapid to leave stranded fæcal matter behind. With too high a velocity of flow, especially in sewers into which grit and gravel enter in considerable quantities, the invert is eroded by the friction.

Minimum Velocity of Sewage in Sewers and Drains.—The minimum velocity to effectively scour a sewer or drain is given by Parkes as $2\frac{1}{2}$ feet to 4 feet per second according to diameter. From this the fall or height required when the drain is flowing full bore may be obtained by Eytelwein's formula.

$$v = 50\sqrt{\frac{dH}{L \times 50d}}$$

v = velocity in feet per second L = length of pipe in feet
d = diameter of pipe in feet H = head or fall of water in feet

EXAMPLE.—What is the fall required to obtain a velocity of 4 feet per second in a 6-inch pipe, 100 feet long ?

$$v = 50\sqrt{\frac{dH}{L + 50d}}$$

$$4 = 50\sqrt{\frac{0{\cdot}5 \times H}{100 + 50 \times {\cdot}5}}$$

$$50 = \sqrt{\frac{0{\cdot}5 \times H}{125}}$$

$$\frac{16}{2500} = \frac{H}{250}$$

$H = 1{\cdot}6$, or 1 foot in $62\frac{1}{2}$ feet.

For the usual velocities in practice Eytelwein's formula

is not considered so exact as Neville's formula, which is as follows for open channels and pipes :—

Let V = velocity in feet per second

R = the hydraulic radius or mean depth in feet

$= \frac{\text{sectional area}}{\text{wetted perimeter}}$ in pipes flowing full or half bore $= \frac{\text{dia. in ft.}}{4}$

S = The sine of the inclination of the pipe

$= \frac{\text{total fall}}{\text{total length}}$

then $V = 140\sqrt{RS} - 11\sqrt[3]{RS}$.

It will be found from this formula that—

4 in. pipes with a fall of 1 in 40 9 in. pipes with a fall of 1 in 90
6 in. ,, ,, ,, 1 in 60 12 in. ,, ,, ,, 1 in 120

running full or half-bore will flow with a velocity of nearly 5 feet per second.

Ganguillet and Kutter's Formula.—It is now generally accepted that the calculated velocity of fluids in sewers and drains will most closely approximate to the actual velocity if calculated by the formula of Chezy.

$$v = c\sqrt{RS}$$

the value of c being taken from the results of the researches published in 1869 and 1870 by Ganguillet and Kutter, of Berne.

$$c = \frac{a + \frac{l}{n} + \frac{m}{S}}{1 + \left(a + \frac{m}{S}\right)\frac{n}{\sqrt{R}}}.$$

The mean values of "n" have been arranged in six categories by Ganguillet and Kutter as follows :—

	Description of channel.	Value of "n."
I.	Channels lined with carefully planed boards or smooth cement... ...	0·010
II.	,, ,, ,, unplaned boards	0·012
III.	,, ,, ,, ashlar or neatly jointed brickwork	0·013
IV.	,, in rubble masonry	0·017
V.	,, in earth (brooks and rivers)... ...	0·025
VI.	Streams with detritus or aquatic plants ...	0·030

EXAMPLE.—Determine the velocity of flow running half bore in a circular pipe 9 inches diameter, with a fall of 12 feet in a length of 1,000 feet, by the formulæ given by (1) Eytelwein, (2) Neville, and (3) Ganguillet and Kutter.

Eytelwein—

$$v = 50\sqrt{\frac{d\mathrm{H}}{\mathrm{L} + 50d}}$$

$$= 50\sqrt{\frac{0{\cdot}75 \times 12}{1000 + 50 \times 0{\cdot}75}}$$

$$= 4{\cdot}65 \text{ feet per sec.}$$

Neville—

$$v = 140\sqrt{\mathrm{R}\,\mathrm{S}} - 11\sqrt[3]{\mathrm{R}\,\mathrm{S}}$$

$$= 140\sqrt{0{\cdot}01199 \times 0{\cdot}1875} - 11\sqrt[3]{0{\cdot}01199 \times 0{\cdot}1875}$$

$$= 5{\cdot}265 \text{ feet per sec.}$$

Ganguillet and Kutter—

$$v = c\sqrt{\mathrm{R}\,\mathrm{S}}$$

$$c = \frac{a + \frac{l}{n} + \frac{m}{\mathrm{S}}}{1 + \left(a + \frac{m}{\mathrm{S}}\right)\frac{n}{\sqrt{\mathrm{R}}}}$$

$$= \frac{41{\cdot}66 + \frac{1{\cdot}811}{0{\cdot}013} + \frac{0{\cdot}00281}{0{\cdot}01199}}{1 + \left(41{\cdot}66 + \frac{0{\cdot}00281}{0{\cdot}01199}\right)\frac{0{\cdot}013}{\sqrt{0{\cdot}1875}}}$$

$$= 80{\cdot}3$$

$$v = c\sqrt{\mathrm{R}\,\mathrm{S}}$$

$$= 80{\cdot}3\sqrt{0{\cdot}1875 \times 0{\cdot}01199}$$

$$= 3{\cdot}807 \text{ feet per sec.}$$

Refuse Disposal.—The removal of "dry" refuse in urban districts is usually carried out by the local authorities, who may by bye-law require the householders to provide dust bins of a suitable non-absorbent material (*e.g.*, galvanized iron) with covers to prevent rain falling upon the contents.

The collection of the refuse is arranged by the authority at definite intervals, sometimes daily, but in any case it should not be less frequent than weekly.

The subsequent disposal is effected by one of the following methods :—

(*a*) Carriage by rail to country districts for tipping on "shoot" and after decomposition spreading on land.

(*b*) Barging to sea, and tipping.

(*c*) Tipping on "shoot" locally.

(*d*) Burning.

It must be borne in mind that decomposing vegetable refuse is a serious danger to health because of the breeding of flies which it facilitates and the well recognised part which flies play in the distribution of disease. For this

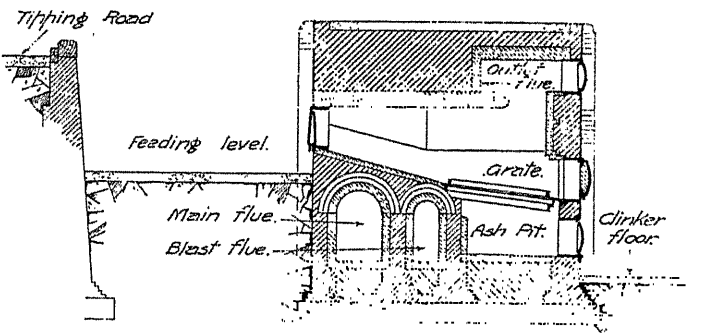


Fig. 838.

reason, any deposit of decomposing refuse near inhabited buildings must be discouraged.

There may be under favourable circumstances some economical value in house refuse, but on hygienic grounds, burning is the only method of disposal which should be approved.

For the burning of refuse, "destructors" are constructed which consist in principle of one or more furnaces so arranged that a high temperature can be obtained sufficient to obtain combustion of the materials composing the refuse. This temperature may be 2,000° Fahr. or more and is obtained by the aid of forced draught caused by a high chimney or by fans.

One type of destructor is shown in Fig. 838. There are

many types which have been developed by specialist firms ; the design of such equipment has become an engineering problem of considerable complexity, to which the addition of steam-raising boilers, etc., has contributed.

The cost which is experienced in disposing of refuse by destructor has militated against the more extensive use of this method, but probably a reversion to simpler types of destructor will assist towards a lower cost.

In rural districts the burning of refuse will offer little difficulty—the kitchen or copper fire, the greenhouse fire, etc., will afford the necessary means. In the case of institutions having considerable quantities of rubbish, small independent destructors may be provided.

CHAPTER XXVI

HOT WATER, HEATING AND VENTILATION

Hot Water Supply and Heating Apparatus.—Of late years it has been found economical to provide apparatus to heat water for domestic purposes and a separate installation for the heating of the building, especially as the heating of a building by a separate installation, after the initial cost, is not only cheaper, but cleaner, more equable, and healthier than by the open fire method.

Methods for the Heating of Buildings.—The heating of buildings is almost exclusively provided for by means of: 1st, low-pressure hot water; 2nd, high-pressure hot water; 3rd, low-pressure steam; or 4th, high-pressure steam.

Methods of Transmitting Heat.—Heat is transmitted from one body to another by one of three processes, or by a combination of two of them, *i.e.*, Conduction, Radiation and Convection. Conduction is the transmission of heat from one particle of a solid to another, or from one solid to another or to gases in contact with it. Radiation is transmission by heat rays emitted from a heated body, which pass through the intervening space without affecting it and warm any bodies that may intercept the rays. The heat from an open fire is a good example of radiant heat, and an illustration of the inefficiency of the open fire form of heating. It is a common experience when sitting in front of an open fire, to become uncomfortably hot on the side exposed to the rays and to feel cold on the side that is in the shadow. Convection is the transmission of heat by the movement of fluids that have been in contact with heated bodies. The air or water becomes heated by

contact, expands and rises, giving place to cooler air or water; this in turn becomes warm and rises, thus setting up convection currents. If the source of heat is maintained, the whole of the air in any room becomes heated. The heating of a room by means of open fires and by radiators is a combination of the radiation and convection processes. An arrangement of heating by radiators placed on the four walls of a room or by heating panels would give the greatest sense of comfort and at the same time admit cool air for respiration.

Hot Water Circulation.—If a vertical tube containing water is heated at the bottom, the heated particles of water will ascend in the centre, rise to the top, become cooled by giving off their heat through the external surface of the tube, and then descend at the sides of the tube towards the bottom.

If a bent tube of the form of a syphon is connected to the upper side of a closed vessel, and the whole is filled with water, and heat is applied to the bottom of the vessel, there will be motion in the water, caused by the convection currents which will be set up. The heated water will rise up one leg of the syphon and return through the other leg into the vessel to be again heated. The direction of the current in the syphon is ensured by connecting the flow end of the pipe to the upper side of the vessel and the return end of the tube to the lower side of the vessel. From the highest point of the syphon a vertical tube or expansion pipe should be fitted to allow the air to escape, and for the expansion of the water and thus prevent damage to the tube or vessel.

Incrustation of Pipes.—Many waters such as those in the London Area contain appreciable quantities of calcium and/or magnesium salts and are known as *hard* waters (see p. 892, *Temporary Hardness*). When such waters are heated carbon dioxide is driven off and the carbonates of calcium and magnesium are deposited and form a hard crust on the interior of boilers and pipes. This crust retards the heating of the water, causes a deterioration of the pipes and requires frequent removal. For the removal of the deposit, or fur, man lids in the boilers, cylinders and tanks,

and screwed connectors in the pipes are provided. Rain-water gives little or no deposit, but owing to acids which it absorbs from the atmosphere corrodes the metal of which the boiler, pipes, etc., are made.

Domestic Hot Water Supply.—The application of these principles in practice is accomplished by one of two methods, the selection of which is determined chiefly by the nature of the water available. The two systems are known as the Direct and the Indirect. These again are divided into the Non-circulating and the Circulating.

The source of heat in all these systems may be from a small boiler behind the firebox in the kitchen range. This method, however, is rapidly becoming obsolete. Apart from the inadequate supply of hot water obtainable, the introduction of gas and electricity for cooking purposes has rendered the range itself obsolete in all small and moderate sized houses. Now invariably an independent boiler is installed to supply hot water for domestic purposes or for heating. With the kitchen range the quantity of hot water available is strictly limited to the boiler supplied with the range. With the independent boiler, obtainable in various sizes, the capacity is proportioned to the quantity of hot water required.

Direct Non-circulating. This system is usually employed for small installations, with probably one bath, a lavatory basin and a kitchen sink. In addition to the boiler, a hot water storage tank of between 30 to 40 gallons capacity is required. This is usually placed on the first floor in a linen cupboard, so that any heat given off by radiation can be usefully employed. A cold water cistern of about 40 gallons capacity is placed usually in the roof in the highest convenient point in order to ensure the maximum pressure. From this a cold water feed pipe is run to the lowest part of the boiler. If the boiler is of cast iron it is preferable to take this pipe to the lowest part of the storage tank, see Fig. 839. From the boiler there is a primary flow and return pipe system. The flow pipe is taken from the highest point in the boiler, and enters the storage tank at about two-thirds its height. The return pipe is taken from the tank at its

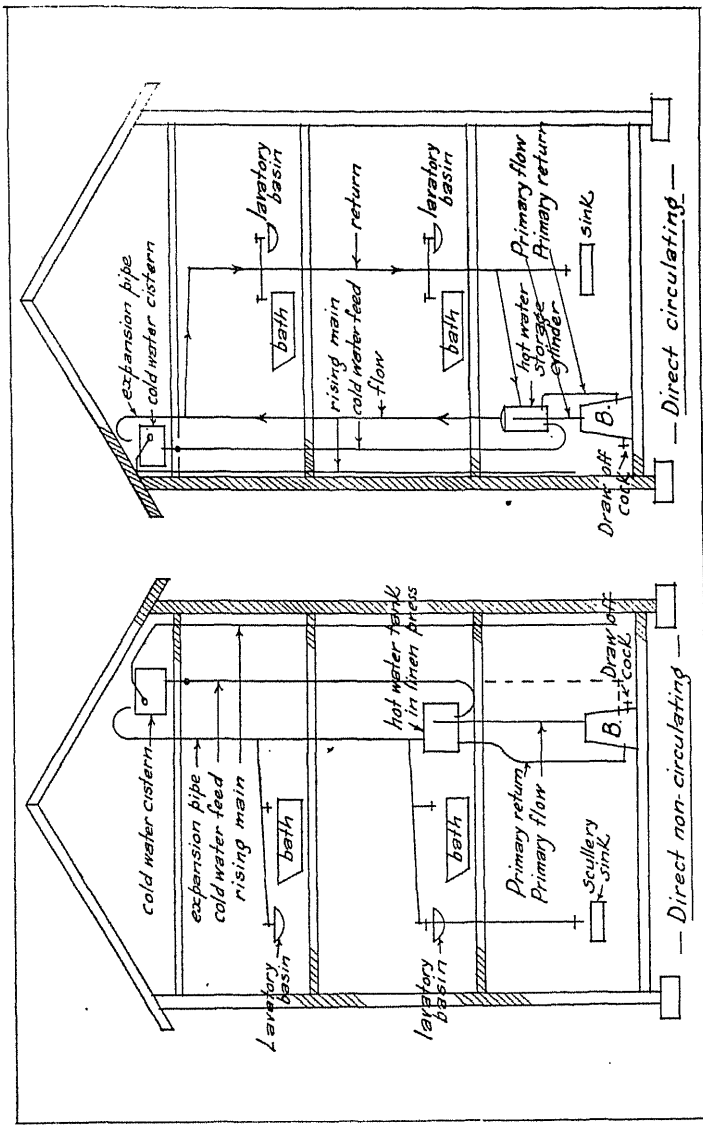


Fig. 839. Figs. 839—840 Fig. 840.

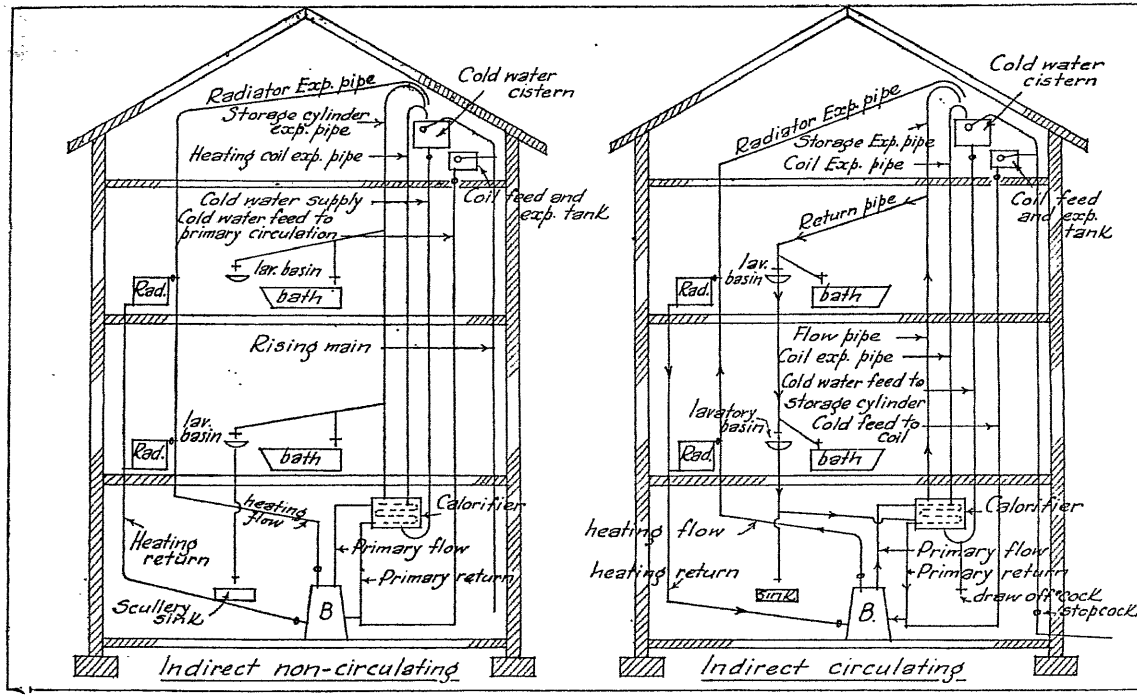


Fig. 841. Fig. 842.

Figs. 841—842.

lowest part, to a point low down in the boiler. From the top of the storage tank a pipe is taken up to the roof; it is turned over the cold water tank and has an open end. This serves as an escape for air, and also to discharge any hot water should the expansion be sufficient to raise it to that level. Above the highest branch connection this pipe can be reduced in diameter. From this point it is known as the expansion pipe. At convenient points branches are taken off this riser to serve the various fitments, see Fig. 839. This system is convenient and efficient where the branches are short. It has the disadvantage that a certain amount of cold water must be drawn off before the hot water discharges. Where the branches are short this is immaterial, the system is simplified, and the cost is reduced.

Direct Circulating.—Fig. 840. In larger installations, or where the bath and other fitments are at a distance from the main flow pipe, it is necessary to have a secondary circulation. This system up to a point is similar to the preceding. In this a storage cylinder is shown as an alternative to the H.W. tank. The cylinder is usually fixed immediately above the boiler and to prevent waste of heat is packed with a non-conducting material and lagged. From above the highest point of discharge a branch return is taken to serve the various fitments and eventually is returned to the cylinder some little distance below the exit of the flow pipe. The return pipe should not be connected far below the exit of the flow pipe, or near to the bottom of the cylinder. If it is, the flow will be partially reversed when the fitting is opened and mixed water will be drawn. At the highest point in the circulation an expansion pipe is taken off and turned over the cold water tank as before.

Where the fitments are distributed in various parts of the building it may be necessary to take more than one flow and return from the cylinder, or branches may be taken from the highest point in the main flow pipes to the various fitments in a series of return pipes. It should be noted that all flow pipes on horizontal runs should be slightly inclined upward, and the returns given a slight fall back to the storage tank; in no case must there be any downward

loop or trap in the pipes, or air locks will occur and the circulation be impeded or stopped. In large or complicated systems it is difficult to maintain this regular arrangement of the pipes. To ensure the circulation a centrifugal pump is introduced on the return pipe where it enters the boiler. The returns from the various circulation systems are connected first to a manifold and passed through the pump into a common return connection. Where a pump is introduced the cold water tank must be sealed and fitted with a safety valve, or the pump will have no effect. The direct systems are suitable for districts where the water is only moderately hard, or where the water is suitable for all domestic purposes without treatment.

Indirect Non-circulating.—Fig. 841. The indirect is a system employed in districts where the water is hard and there is liable to be a large deposition of scale in the pipes, and also where, in moderate sized houses, it is required to instal some radiators to heat parts of the building. From the boiler a primary circulation is taken to a heating coil placed within the storage cylinder. The water contained in the coil and the boiler is small in quantity. It is not changed or used except for the small amount lost by evaporation, and the amount of scale precipitated is small and takes place within the primary circulation. The temperature of the water in the storage cylinder that encloses the coil never rises above 130° Fahr. Precipitation does not take place to any extent until the water is above 150° Fahr. Thus there is no danger of the pipes in the secondary supply becoming furred. As the water used for domestic purposes rarely exceeds 120° Fahr., there is no advantage in raising its temperature above that point.

In the indirect system it is necessary to have a cold water feed and expansion pipe to the primary circulation. The cold water feed is taken from an expansion tank, see Fig. 841, the pipe being connected to the return from the coil and entering the lower part of the boiler. A separate cold water feed is provided for the secondary supply, the cold water feed pipe being taken in at the bottom of the cylinder. The flow pipe is taken out of the top of the cylinder, direct to the roof and discharges over the cold

water cistern if the level of the heated water should rise to that level. Branches are taken at convenient positions on this pipe to the fittings required, see Fig. 841. As in the direct system there will be a quantity of cold water drawn off before the hot appears, but if the fittings are near the main flow this is of little consequence.

If it should be required to heat portions of the building by radiators, a separate pair of circulating pipes should be taken from the boiler direct, see Fig. 841. Where the heating points are scattered to different parts of the building several pairs of circulating pipes may be required; under these conditions one main flow is taken into a manifold from whence the flows to the different circulations are taken. In a like manner all the returns are taken to another manifold from whence a single return is connected to the boiler. To ensure a good circulation a pump or turbine electrically propelled can be inserted in the main return pipe; a pump of this kind in a circulating system not only ensures a good circulation but also enables a reduction in the pipe sizes to be made.

Indirect Circulating.—Fig. 842. This is similar to the direct circulating system. A branch connection is made to the main flow above the highest fitting and is continued back to the boiler so that there is a continuous circulation of the water. Short branch connections are made to this main return pipe so that hot water is discharged immediately the taps are turned on. The same observations that were made in the paragraph on the direct circulating system with regard to several circulations apply here. With reference to a heating system being connected to the installation, the arrangements are similar to that described in the preceding paragraph.

In all these systems a safety valve should be fitted, attached to a short branch just above the storage cylinder.

Heating of Buildings.—There are two general systems commonly employed for the heating of buildings by low-pressure hot water: first, that known as the "drop system," and, secondly, the "two-pipe system."

No hot water is drawn off from the pipes of the heating

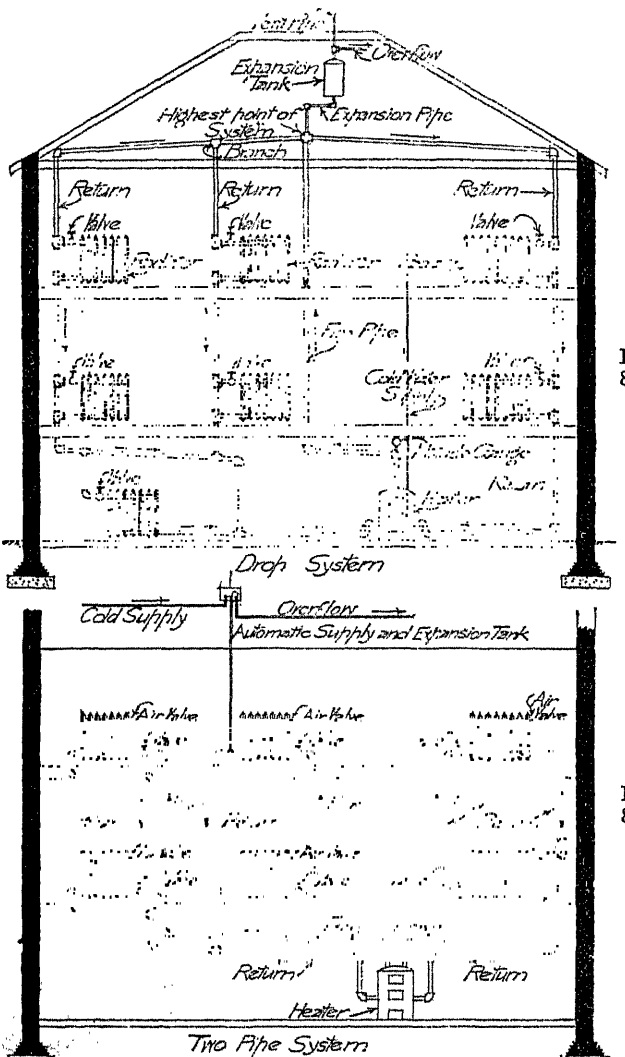


Fig. 843.

Fig. 844.

Figs. 843—844.

installations, consequently there is very little waste of water, and the pipes are thus prevented from being encrusted with fur.

" *Drop-System.*"—The temperature of the water is raised in a heater placed in the lowest part of the building ; then by the first method it is taken in a flow-pipe to the highest part of the building required to be heated. At this point it is distributed by as many branch pipes as are required ; these run vertically downwards, and join the return pipe at the lowest part of the system. Radiators are connected to these return pipes at the various levels, the circulation through the radiators being controlled by a valve. The system is shown in Fig. 843. At the highest point in the system an expansion pipe, including a tank and overflow, is fitted, to relieve the system of pressure due to the expansion of heated water.

" *Two-Pipe System.*"—In this arrangement as many flow-pipes as are required are taken direct from the heater to each system of radiators. The heated water is passed from the flow pipe to the bottom of the radiator through which it circulates, and passes into the return pipe on the opposite side of the radiator. The supply to the radiator is controlled by a valve on the branch flow pipe. An air valve must be supplied at the highest point in each radiator to draw off the air which, in this system, collects in the pipes and retards circulation. An expansion pipe is taken from the highest point in the system. Fig. 844 shows the expansion pipe provided with an automatic supply and expansion tank.

Of the two systems in compact buildings the " drop " is preferred. As the hot water mainly gives up its heat in the radiators on the return it causes a better circulation, and as the air does not collect in the pipes no air-valves are required. The two-pipe system would be preferable in straggling buildings, where the distances between the radiators would be great.

In large buildings heated by hot water, the circulation is assisted by mechanical means. Pumps and turbines, actuated by steam or electricity, are fitted to either system

and assist the circulation, and are specially useful in overcoming the resistance of sinkings or syphons in a system of pipes.

High-Pressure Systems for Domestic Purposes and the Heating of Buildings.—The small bore system is shown in Fig. 845, and consists of a boiler composed of a coil of strong wrought-iron pipe screwed together with sockets and having right and left threads, the pipes being $\frac{7}{8}$ inch bore, and $1\frac{5}{16}$ inch outside diameter, and having a total content equal to about one-sixth of the entire apparatus. The pipe coil is set in a fire-brick furnace, fixed in the basement of the building, and is connected with similar pipe to the fittings on the various floors. On the various floors coils of pipe are inserted in cold water tanks, the contents of which they heat. Hot water is drawn from the tanks to supply the baths and sinks. The return pipe is dipped at its lowest part to prevent the circulation acting backwards. The heating pipes, which contain about 1 gallon of water to each 40 feet run, are filled and hermetically sealed, so that no waste or evaporation of water can take place. Provision is made for the expansion of the heated water by an expansion chamber, the capacity of which equals about one-tenth to one-twentieth the contents of the entire apparatus. The whole of the pipes are tested to a pressure of from 2,000 to 3,000 lbs. per square inch.

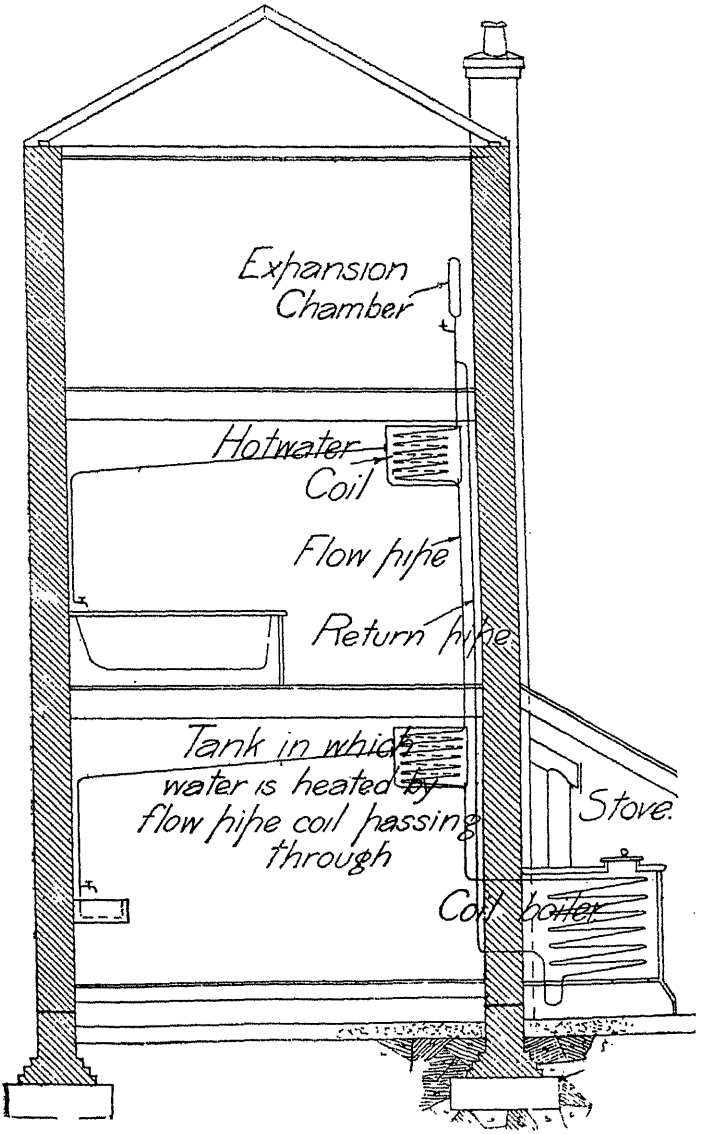
A very high temperature can be produced with this apparatus, the temperature varying with the pressure exerted by the excessive heating, from 200° to 500° Fahr.

The temperature can be raised so high that it is always advisable to fix the pipes some little distance from any woodwork, or place asbestos round the pipes when in contact with any inflammable material.

This system is practically obsolete owing to the danger in the event of leakages.

Steam, Low Pressure.—This system of heating consists of a boiler, usually tubular, placed in the basement. From the boiler wrought-iron pipes are conducted to the various rooms, and there connected to coils of wrought iron or

HOT WATER SUPPLY AND WARMING,
"HIGH PRESSURE" OR "SMALL BORE SYSTEM."



VERTICAL SECTION
Fig. 845.

copper, or to cast-iron radiators; return pipes from the radiators are taken back to a steam trap, the overflow from which is taken to a sump and pumped back into the boiler or returned to the boiler by a steam injector.

The pressure at which a low-pressure steam apparatus works is from 5 to 10 lbs. per square inch, and the arrangement requires careful fitting and constant attention in order to maintain a uniform heat and to avoid accidents.

Steam, High Pressure.—High-pressure steam is usually supplied direct from a steam boiler, or from the exhaust of a steam engine. The working pressure varies from 10 to 50 lbs. per square inch. The steam is conveyed by wrought-iron pipes to coils or radiators placed in the positions to be warmed, and which have been made strong enough to safely withstand the pressure exerted.

Warming by steam, low and high pressure, by the direct, indirect, or combined methods, is often used for workshops, drying rooms, public buildings, and large establishments (used for manufacturing purposes), and also for producing dry air at high temperatures, as in Turkish baths. These installations should always be under skilled supervision.

As the heat from steam radiators is quickly diffused in the air, the ventilating arrangements in rooms should be under good control for either opening or closing. A building can be quickly warmed by steam, but as the steam quickly condenses when the boiler fire is damped, it follows that there is no stored heat in the radiators, as is the case with hot water heating systems.

In steam heating installations noises are often heard which are very objectionable, through the condensation which takes place in the pipes. But these noises can be avoided by carefully aligning the pipes and making provision by relief valves or otherwise disposing of the condensed water.

Best Method for General Purposes.—The "low pressure" hot water heating system is the best for small buildings and residences, also for large buildings if assisted by mechanical means, for the following reasons: It can be applied to any

description of building, public, private, horticultural or manufacturing; the heat given off is healthy, mild, and agreeable, the temperature of the pipes seldom exceeding 190°, and never 212° Fahr.; consequently the air which comes into contact with them is gradually warmed and not scorched, and the temperature in the pipes can be gradually raised or lowered, or evenly maintained. The small amount of fuel and labour required, and the freedom from the generation of those gases which are injurious to plants, and the quality of the heat given off, render low pressure hot water heating systems the most satisfactory of all for ordinary use. For heating dwelling houses, the heating coils, or radiators, should be fixed in the entrance hall near the staircase, in the corridors and passages, reception rooms, and in bedrooms under the window, care being taken to efficiently ventilate the latter. Open fires may thus be superseded and the contamination of the atmosphere by domestic smoke would be enormously reduced.

Where excessively high temperatures are required, superheated water or steam installations give the most satisfactory results.

Panel Heating.—Panel Heating is a method of heating, usually by low-pressure hot water, circulating through coils embedded in the ceiling or walls of a building. This method heats the walls and panels of the ceilings of a room direct. This method is of comparatively recent introduction. The opinion of experts is not unanimous, taking all things into consideration, as to the advisability of installing it in all types of buildings. The action of the heated pipes on the concrete in which it is embedded, the risks of the plaster cracking and falling away due to the unequal expansion and contraction of the plaster and the metal pipes, and the possible discoloration in the panels; these points must be considered. Doubtless these disadvantages will in course of time be overcome and then the absence of visible piping and radiatiors will constitute a distinct improvement from the decorative standpoint.

Strength of Pipes.—Pipes under hydraulic pressure frequently fail and break with a longitudinal fracture.

Their ultimate resistance may be calculated by the formula—

$$p \times d \times l = 2\ ltf_t$$

when

p = pressure per unit of area upon diametral plane
l = width of diametral plane as shown in figure 846
d = length of diametral plane
t = thickness of pipe
f_t = ultimate resistance in tension or safe resistance to tension and values of which for cast iron, wrought iron and steel are given in chapter on steel.

EXAMPLE.—To what head of water could a 12-inch cast-iron pipe of $\frac{5}{8}$-inch metal be exposed without the stress on the metal exceeding 2,000 lbs. per square inch? Then by formula—

$$p \times d \times l = 2\ ltf_t$$

$$p = \frac{2tf_t}{d}$$

$$p = \frac{2 \times \frac{5}{8} \times 2000}{12}$$

$p = 208\frac{1}{3}$ lbs. per square inch.
$= 208\frac{1}{3} \times 16$ oz. per square inch

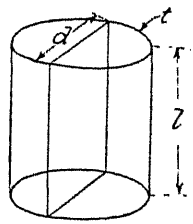


Fig. 846.

Let h = head of water in inches
w = weight of cubic inch of water

$$= \frac{6\frac{1}{4} \times 8 \times 20}{1728} = \frac{1000}{1728} \text{ ozs.}$$

then

$$hw = p$$

$$h = \frac{p}{w}$$

$$= \frac{208\frac{1}{3} \times 16 \times 1728}{1000}$$

$= 5760$ inches
$= 480$ feet.

VENTILATION

Ventilation is the removal of all vitiated air from a building and its replacement with fresh. Where this vitiation is the result of human agency it means the removal of the air charged with the products of respiration and the water vapour given off from the bodies and breath of the inmates.

Buildings to merit the term healthy should satisfy the following conditions: (1) The internal air should be pure and constantly changed by overflowing currents so diffused as to avoid "draughts"; (2) the building should be clean and dry; (3) no decomposition of the building materials should be taking place, and it is especially important that all woodwork should be exposed to a current of dry air; and (4) all soil and waste pipes and drains should be ventilated and trapped. All the above except No. 1 have been fully dealt with; the first will now be discussed.

Pure Air Standard.—The degree of purity of the air is determined in most instances according to the quantity of carbon dioxide present, the standard of external atmospheric air being taken as containing not more than 0·04 per cent. of CO_2.

Carbon dioxide is taken as the measure of impure atmosphere, not that by itself it would be so harmful, but because under normal circumstances the amount of CO_2 may be taken as indicating the approximate quantity of the more dangerous gas exhalations from animal bodies.

Air within buildings containing 0·06 per cent. of CO_2 may be considered vitiated, but with 0·09 or 0·1 per cent. it becomes stuffy and unbearable. It becomes evident, therefore, that the air in habitable rooms should never contain more than 0·06 per cent. of CO_2. The air should be kept in this condition by ventilation and without any draught perceptible to the senses being occasioned, the greatest permissible velocity of the air current being 3 feet per second, when the temperature is between 50° and 60° Fahr. At lower or higher temperatures the same rate of motion would cause noticeable draughts. Pure air in buildings is necessary for the sustenance and improvement of

health, for the perfect combustion of fuel, and for the preservation of the materials of which the buildings are constructed.

Oxygen Supply for Fuel.—If a chamber has sufficient outlets independently of the fireplace, the necessary oxygen

Fig. 848.

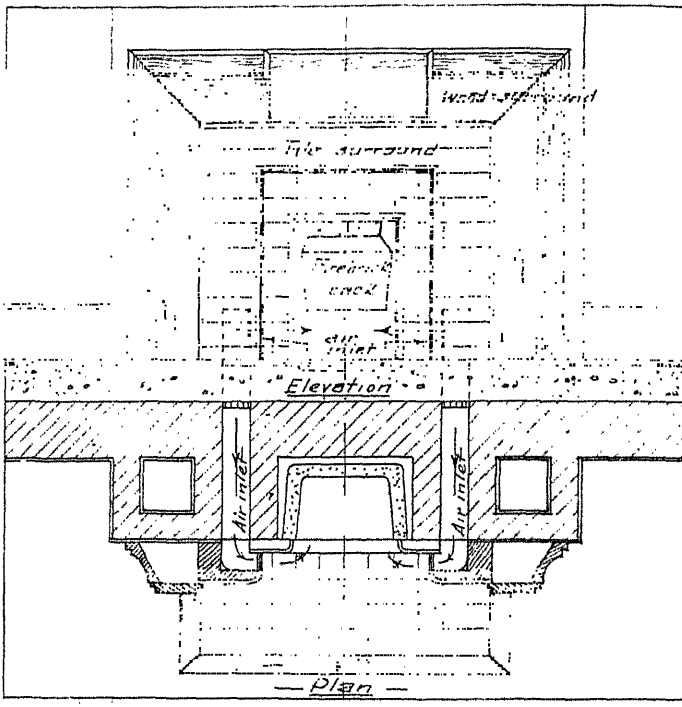


Fig. 847.

Figs. 847—848.

for the combustion of fuel is best supplied to the fire direct from outside the house by special air ducts discharging through the fireplace jambs, as shown in Figs. 847 and 848, the supply being regulated by a sliding valve. Where

provision is not made for supplying the fire and the draught up the chimney, the air is drawn through the crevices of imperfectly fitting window sashes and doors, and this often causes unpleasant draughts.

Density of Gases.—The comparative densities of the gases contained in and which in many cases pollute atmospheric air are—

Pure atmospheric air	14·4
Carbon dioxide (CO_2)	22
Water vapour (H_2O)	9
Ammonia gas (NH_3)	8½
Nitrous acid (HNO_2)	23½
Sulphuretted hydrogen (H_2S)	17
Sulphur dioxide (SO_2)	32

Sources of Atmospheric Impurities.—Carbon dioxide is given off in the breath of mankind and all living animals, in the combustion of fuel, in the burning of gas, etc., and in the decay of organic matter.

Water vapour arises from damp soils, marshy lands, lakes, ponds, and streams, and is harmful only when the air is overcharged with it, causing conditions favourable to colds, chills, rheumatism, etc. Water vapour is also given off in respired air.

Ammonia is formed during the decomposition of moist animal and other matters, which contain hydrogen and nitrogen, and is very noticeable in unventilated stables, cow byres, piggeries, and urinals. Sulphur dioxide will be found in rooms where coal gas or coke is burning, and in the neighbourhood of gas manufactories.

Sulphuretted hydrogen is given off from decaying animal and vegetable matter, and is produced in large quantities in cesspools and in unventilated sewers and drains.

Rise and Fall of Gases.—The movement of air in a room is influenced largely by its composition and temperature. In its natural state the air contains certain impurities, and during its contact with the lungs it obtains a proportionately considerable increase of impurities—carbonic acid, aqueous vapour, etc.—as well as obtaining an increase of temperature.

The weight of air after respiration as compared with air before inhalation, volume and temperature being equal, is *greater* by about 2 per cent., but the weight, volume for volume, of air at the time of exhalation is *less* than air before inhalation by nearly 5 per cent., owing to the increase of volume, and consequent decrease of density, due to the increased temperature at which air is expelled from the lungs.

Due to these natural causes it is apparent that the expired breath first has a tendency to rise on account of its higher temperature and consequent lower density, but, subsequently, as its temperature falls again to that of the fresher air of the room the respired air will, owing to its greater density, tend to fall, and the impurities which it contains may, under certain circumstances, be collected at a lower level.

In addition to the heating effects of breathing, air is heated by fires, artificial illuminants, and the bodily heat of the occupants, and, in consequence, the air near the ceiling of a room is usually found to be at a higher temperature than near the floor.

These conditions will need consideration in the provision of the inlets and outlets for any system of ventilation, natural or artificial.

Ventilation of internal spaces is usually classified as Natural and Mechanical.

Natural.—Where the air in a building is changed by the opening of windows, by the provision of a system of inlets and outlets, where reliance is placed for changing the air on the difference in temperature and weight of the outside and inside air and the consequent setting up of currents, or any other means that requires no mechanical contrivance to assist or cause the air currents ; this method of ventilation is known as " natural ventilation."

Mechanical.—Where the air currents are caused by mechanical means and air is supplied and exhausted by fans, the process is known as " mechanical ventilation."

Satisfactory results may be obtained in most cases of

residential work by the natural method or where the number of people in any chamber is small compared with its cubic capacity. In all cases where large numbers of people are congregated, such as churches, theatres, cinemas, or factories etc., natural methods, over which there is no control, are inadequate, and mechanical means are necessary to provide the requisite amount of air to secure healthy conditions. Heating and ventilation, although two separate subjects, must be considered together. The maintenance of any given temperature in a room, is dependent upon the adequacy of its heating elements to make good the heat losses by conduction through walls, windows, etc., and by the removal of large quantities of heated air for the purposes of ventilation. The latter constitutes one of the greatest sources of loss and must therefore receive primary consideration, when designing radiator areas, boiler capacities, etc.

Natural Ventilation.—This may be carried out by means of the open window. If the outside air is colder than the inside, and the open sash is near the ceiling it will usually act as an outlet. If no special inlets are provided, there will be draughts from doors, etc. The current may be reversed if a good open fire is functioning, when the air to support combustion may be drawn from the open window and from the crevices about the doors. If the internal and external temperatures are about the same, the vitiated air is modified by diffusion. A certain amount of fresh air may be obtained by the ventilating bead, or by arranging openings above the sashes as shown in Fig. 849. Air inlets of the type known as Tobin tubes may be provided, see Figs. 850 to 851. These consist of air gratings through the external walls and flues arranged in the thickness of the wall, or formed by wood or metal ducts arranged on the inner face of the wall. These are usually arranged to discharge the incoming air at a height of about 5 feet.

In large apartments exhaust currents may be made through ducts formed above grids arranged in the ceiling or high up in the walls. The ducts lead to ventilating turrets formed on the roof. Currents are induced by gas jets, either in the duct, or grouped as in the old-fashioned sun

burner, see Fig. 852. Wherever exhaust currents are contrived, inlets should be provided to supply fresh air

Fig. 849. Fig. 850.

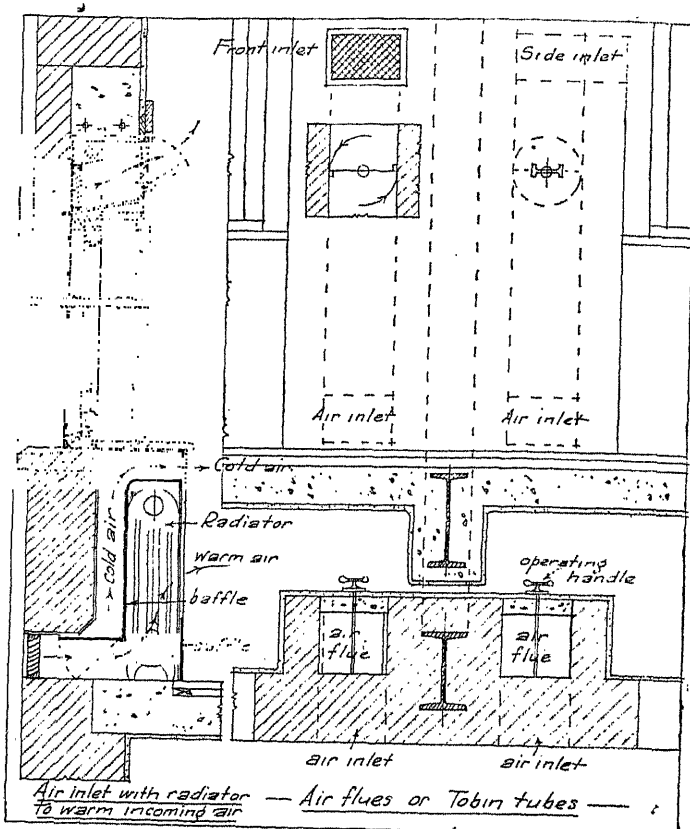


Fig. 851.

Figs. 849—851.

from a known source or the supply may be vitiated air from other parts of the building, through the doors.

Where no open fireplace is provided in any habitable

room the Bye-laws in most districts demand that an air grating communicating with the open air be provided.

Mechanical Ventilation is any system where the air in any enclosed space is changed by mechanical means. In

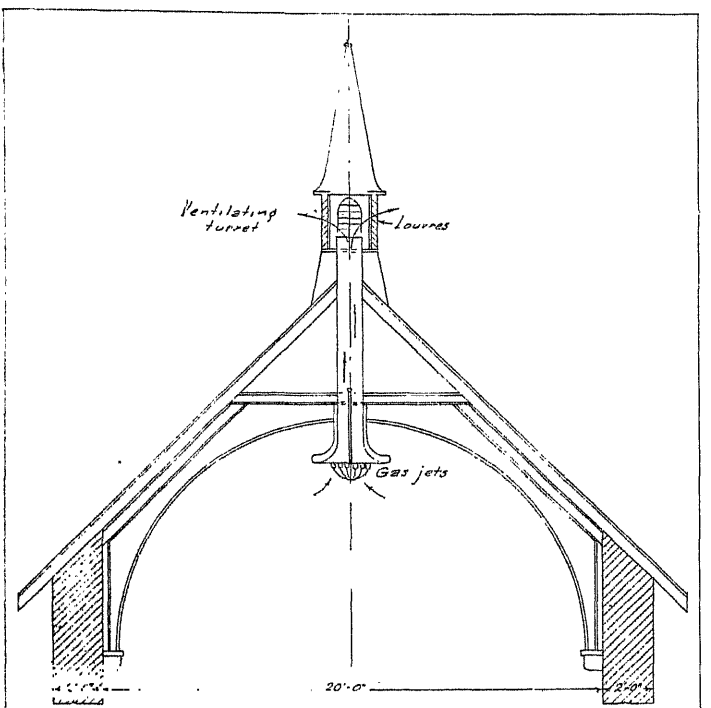


Fig. 852.

any such system there must be both inlets and outlets properly proportioned in order to supply the calculated quantities of fresh air, at any given rate of extraction or propulsion. The proper working of such systems are naturally affected by the opening of doors and windows. If the latter are kept closed, and doors are closed after using, any

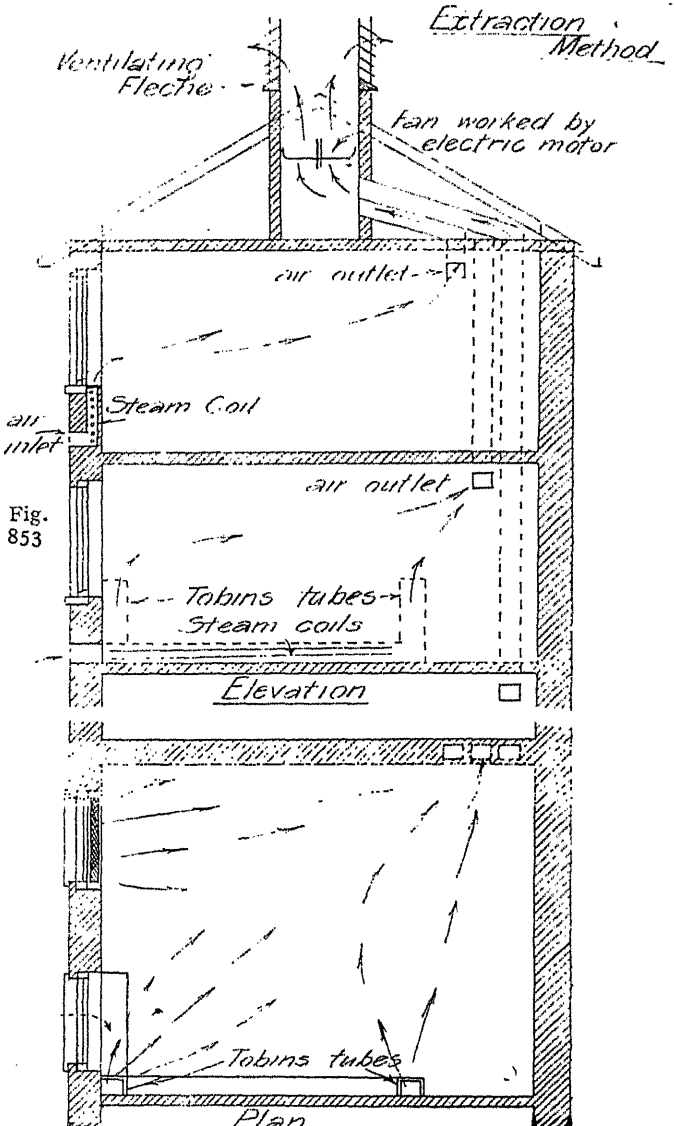


Fig. 853

Fig. 854.

Figs. 853—854.

disturbance in the proper working of the system will be only temporary.

There are three systems of mechanical ventilation.

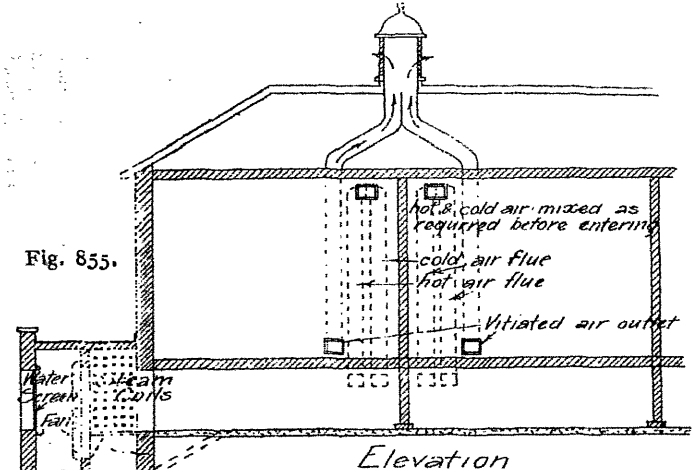
1. The Extraction.
2. The Plenum.
3. A combination of the former two. The Plenum Extraction System.

(*a*) In the extraction method, as illustrated in Figs. 853 and 854, air inlets are formed, discharging fresh air into the rooms at a height of from 4 to 6 feet through Tobin tubes, and the outlet is arranged within a foot of the ceiling on the opposite side of the room from which the air enters. The air, after traversing the room, is drawn through the outlet, usually by a fan driven by mechanical means, and conducted to the roof and discharged above the highest level, preferably through a ventilating turret.

Frequently a coil of steam or hot water pipes is arranged in the inlets, the fresh air thus being heated before entering the rooms. For efficiency by this method the outlet should be arranged in one angle of the room, and there should be an inlet at each of the remaining three angles.

Plenum.—By this system an air inlet is selected on that side of a building where the purest air is available. In the aperture is fixed the screen or filter, usually fixed in an iron frame. There are various forms of these either fine gauge screens, or a series of baffle plates. A constant stream of water is kept flowing down the screen or a series of atomisers giving a fine mist of water through which the air is drawn by means of a blower fan. The action of the screens is to remove all mechanical impurities from the air. The air should leave the filter clean and pure. At this point it may be further disinfected by the introduction of ozone from an ozonizing apparatus; a form of an electrical condenser in which ozone, free from any of the compounds of nitrogen, is discharged into the air stream. The air may be forced through a battery of hot water tubes and be heated before being forced into the various rooms. A cold duct is provided so that the air may be delivered either cool

Fig. 855.



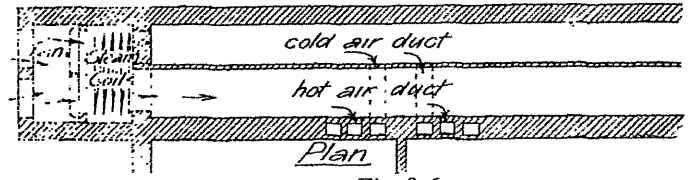


Fig. 856.

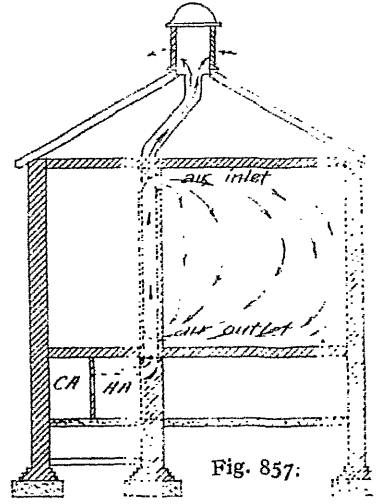


Fig. 857.

Injection Method

Figs. 855—857.

or of any temperature up to the limit of the heating coils, see Figs. 855 to 857. In the distribution of the air, it is essential to employ properly formed sheet iron ducts with properly dimensioned branches, and at all changes of direction sharp angles must be avoided and sweeps as large as the circumstances will permit be provided, to reduce the friction on the sides of the ducts and prevent eddies; inattention to this point will lessen the efficiency of the system. Mechanical ventilation designed to give definite results is a highly specialist business. Every case presents its own problems and all the factors must be carefully considered and provided for if the installation is to be successful.

The Plenum extraction.—In complicated installations the plenum system frequently requires to be supplemented by extraction fans, as in cases where the delivery of the pure air is likely to be sluggish, or where it is desired to discharge vitiated air, containing obnoxious fumes, as from kitchens or various manufacturing processes, in specially isolated areas.

It is usual in this country to measure the quantity of heat by the British Thermal Unit. This is the amount of heat that will raise the temperature of one pound of water one degree Fahr.

There are two main causes of the loss of heat from a building. (1) That lost by conduction through the fabric of the structure and (2) that lost by the expulsion of heated air for the purposes of ventilation. Subjoined is a list of the losses, through walls, windows, etc. The loss due to the removal in bulk of the air from any chamber, when the outside temperature is 30° and the inside is 60° Fahr., is approximately 0·57 B.T.U.'s per cubic foot.

Then to decide upon the quantity of heat to be supplied to any space and knowing the cubic contents and the total heat losses, the amount of radiation to make good the losses can be obtained.

Thus the sum of the heat losses for every room and corridor in a building will give the rating of the boiler. There are so many conditions affecting the temperature of any given room and of the system collectively, that considerable experience alone can gauge with any degree of

accuracy the contingent allowances that must be made to ensure not only efficient but also economical working of any installation.

Heat losses by passing through walls, etc.: the following factors are the loss in B.T.U.'s per hour, per degree Fahr., per square foot.

½ Brick	...	...	0·5
1 ,,	...	...	0·35
1½ ,,	...	...	0·27
2 ,,	...	...	0·23
2½ ,,	...	...	0·2
3 ,,	...	...	0·17
Sandstone			
24″ thick	...	...	0·34
30″ ,,	...	...	0·3
36″ ,,	...	...	0·23
Glass single	...	...	1·07
,, doubles	...	...	0·6
Wood doors	...	...	0·45
Ceiling boarded and tiled roof	...	...	0·4
Boarded flat roof, lead and plaster	...	...	0·17
6″ concrete flat asphalted and plastered	...	...	0·28
Limestone			
24″ thick	...	...	0·36
30″ ,,	...	...	0·32
36″ ,,	...	...	0·30
Concrete			
4″	...	...	0·6
6″	...	...	0·55
8″	...	...	0·53
Wood floor, plastered ceiling	...	...	0·13
,, ,, pugged			0·1
6″ concrete, wood floor, plastered ceiling	...		0·2
Corrugated iron roof	...		1·80
Skylights	...	...	1·1
Double Skylights	...		0·5

For walls with north or east aspect add	...	...	15	per cent.
,, ,, west ,,	...	...	5	,,
Heating during day only	...	...	15	,,
Heating at long intervals	...	...	50	,,

All rooms over 12 feet high, the quantity of heat should be increased by the following percentages :—

12 feet to 15 feet	...	...	10 per cent.
15 feet to 20 feet	...	...	20 ,,
20 feet upwards	...	...	30 ,,

SCHEDULE OF INTERNAL TEMPERATURES

Sitting-rooms	60° F
Bedrooms	55°
Staircases, corridors, etc.	50° to 55°
Shops	55°
Workshops, sedentary	60°
,, other	50° to 55°
Offices	60°
Bath rooms	60°

Rooms.	Cubic contents.	Air changes.	Glass.	Walls.	Floor.	Roof or ceiling.	Total.	Radiation.	Piping.
			1·07	14″ Brick 0·27	0·1	0·4		150° F. — 60° = 90° diff. Emission approx.	
Two Windows 6′0″×3′6″. Outside Temperature 30°. Inside Temperature 60°. Floor to ceiling 12 ft.	20′×15′×12′ 3600 cub. ft.	12 persons at 1200′/hr. = 14400 cub. ft. 4 changes 14400×0·57 = 8208 B.T.U.'s	Area 2/6′0″ × 3′6″ 42′0″ 42×1·07×30 1350 Add for easterly exposure 15% = 198 1350 198 1548	S. 240 E. 138 W. Heated R. N. ,, ,, S. 240 × ·27 = 65 B.T.U.'s E. 138×0·27+15% = 37 + 5·5 = 42·5 65 42·5 107·5	Heated Room beneath Nil.	Ceiling Roof boarded and tiled 300′ sup. 300×·4 = 120 B.T.U.'s	8208 1548 107 120 9983 Add 50% for heating from cold. 9975 4988 14963 B.T.U.'s required.	140 B.T.U.'s per sup. ft. per hour 14963 140 = 107 sup. ft. for radiation No. 2/16 section 2 column radiators 2′ 8″ high 4′ 0″ long to be placed under windows.	*Note.* Details of piping depend upon many factors, height travel, bends, etc., and could only be given with a complete case.

The general principles of estimating the area of radiation required is given in the preceding table. The case of one room only is taken. Each room and corridor or space heated is estimated in a similar manner, and a summary is made. The total quantity of heat required will give the necessary data for determining the boiler required.

EXERCISES
ON EXAMINATION PAPERS.

Note.—The numbers refer to questions given on pages 1003 *to* 1121.

MATERIALS

Nos. 3, 8, 19, 20, 21, 22, 23, 27, 34, 35, 41, 49, 51, 52, 53, 59, 130, 131, 132, 133, 134, 135, 141, 144, 146, 158, 190, 191, 218, 233, 238, 239, 265, 266, 267, 268, 269, 298, 321, 324, 345, 346, 370, 388.

FOUNDATIONS

Nos. 30, 70, 104, 105, 154, 352, 360, 379, 397, 434.

BRICKWORK

Nos. 7, 28, 37, 54, 78, 99, 106, 140, 142, 156, 157, 177, 221, 226, 234, 269, 287, 289, 315, 321, 333, 367, 382, 383, 390, 417.

FLUES AND FIREPLACES

Nos. 2, 9, 33, 38, 39, 48, 203.

MASONRY

Nos. 248, 290, 301, 338.

CARPENTRY

Nos. 57, 60, 61, 62, 72, 73, 80, 137, 138, 145, 146, 148, 151, 159, 178, 185, 204, 206, 222, 229, 241, 251, 252, 279.

RIVETING

Nos. 95, 96, 273, 307, 404.

GRAPHIC STATICS

Nos. 276, 290, 304, 306, 328, 381, 412, 425.

PILLARS AND STRUTS

Nos. 70, 71, 172, 173, 258, 280, 283, 316, 318, 327, 339, 340, 356, 357, 358, 377, 389, 391, 400, 419.

THEORY OF CONSTRUCTION

Nos. 64, 92, 123, 171, 209, 271, 277, 308, 317, 327, 328, 347, 348, 349, 363, 366, 378, 384, 386, 399, 401, 402, 403, 412, 413, 431.

GIRDERS

Nos. 43, 92, 94, 112, 125, 126, 127, 174, 176, 193, 212, 214, 241, 254, 270, 274, 275, 278, 281, 288, 291, 293, 295, 302, 303, 309, 325, 326, 329, 330, 341, 351, 359, 361, 362, 364, 369, 373, 375, 380, 383, 385, 387, 392, 394, 396, 400, 406, 407, 409, 414, 415, 416, 418, 420, 422, 423, 426, 427, 428, 430, 438.

REINFORCED CONCRETE

Nos. 12, 97, 98, 111, 124, 128, 210, 211, 253, 255, 257, 259, 282, 284, 285, 286, 294, 296, 313, 314, 331, 336, 337, 350, 353, 354, 355, 358, 365, 371, 372, 374, 389, 390, 398, 405, 432.

ROOFS

Nos. 11, 69, 74, 79, 100, 109, 129, 163, 164, 175, 181, 189, 196, 205, 213, 214, 244, 249, 256, 272, 299, 305, 322, 369, 381, 393, 408, 425, 437.

JOINERY AND STAIRCASING

Nos. 1, 4, 13, 14, 36, 55, 75, 81, 82, 101, 102, 107, 136, 147, 149, 150, 155, 161, 180, 182, 186, 187, 188, 197, 198, 202, 216, 219, 228, 230, 231, 232, 237, 240, 245, 246, 247, 300, 323.

HOT WATER AND VENTILATION

Nos. 24, 25, 26, 50, 83, 84, 85, 86, 87, 88, 118, 119, 120, 121, 122, 164, 165, 166, 224, 261, 262, 264.

SANITATION AND WATER SUPPLY

Nos. 16, 17, 18, 40, 46, 47, 76, 89, 90, 91, 113, 114, 115, 116, 117, 167, 168, 169, 170, 208, 217, 220, 222, 223, 236, 242, 260, 264, 297, 320, 342, 433.

MISCELLANEOUS

Nos. 5, 6, 10, 15, 29, 31, 32, 44, 45, 60, 63, 65, 66, 67, 68, 77, 103, 108, 139, 143, 153, 160, 162, 179, 183, 184, 186, 192, 194, 195, 199, 200, 201, 207, 215, 225, 227, 235, 243, 250, 263, 310, 311, 312, 317, 319, 324, 332, 333, 334, 335, 336, 346, 366, 376, 382, 383, 384, 395, 410, 411, 421, 424, 429, 433, 435, 436.

APPENDIX

OF EXAMINATION QUESTIONS SET AT RECENT PAPERS BY VARIOUS BUILDING INSTITUTIONS

EXAMINATION QUESTIONS

NOTE OF ABBREVIATIONS.

C.S.I. = Chartered Surveyors' Institution.
I.O.B. = Institute of Builders.
I. Struct. E. = Institution of Structional Engineers.
R.I.B.A. = Royal Institute of British Architects.

1. The staircase from ground to second floor is constructed in reinforced concrete with *in situ* terrazzo finish—1 inch thick to treads with moulded nosings and $\frac{3}{4}$ inch thick to risers. The balustrade is in built-up framing covered both sides with veneered blockboard and has a moulded capping as handrail. The walls are panelled to balustrade height with material to match. To 1-inch scale draw a plan of five steps including commencement of landing, and sections through the same showing the balustrade and wall panelling and the reinforcement in the stairs. The detail is to be fully dimensioned.

I.O.B. 1939.

2. The chimney shaft from the basement upwards has a $4\frac{1}{2}$-inch firebrick lining with 9 inch common brickwork around. To $1\frac{1}{2}$-inch scale draw two successive courses showing the bonding of the brickwork of the shaft and the junction with the party wall. The flue is to be 14″ × 9″ in the clear.

I.O.B. 1939.

3. Give a brief account of the decay and preservation of timber. Describe with sketches examples of construction conducive to such decay. Suggest improvements in construction to avoid it and measures to remedy it where it has occurred.

I.O.B. 1939.

4. Illustrate the following with sketches :—

(i) Joint of middle rail with diminishing stile of door.
(ii) Secret nailed joint in wood strip flooring.
(iii) Damp courses to 9-inch parapet wall with stone coping and asphalt flat.
(iv) Joint in stone cornice.
(v) Cramp in stone cornice.
(vi) Secret fixing of wall panelling.
(vii) Threshold of external door, showing protection against driving rain :—
  (*a*) door opening inwards.
  (*b*) door opening outwards.

I.O.B. 1939.

5. State causes for the following failures and discuss measures to prevent them :—

(*a*) Cracking of terrazzo paving laid *in situ*.
(*b*) Lifting of floor tiling.
(*c*) Blistering of paintwork. I.O.B. 1939.

6. Name the chief types of glass employed in building work and their appropriate uses. Describe their characteristics and decorative factory processes which may be applied to them. I.O.B. 1939.

7. Describe and illustrate with sketches the work of underpinning a 3 ft. 0 in. brick wall in brickwork to a depth of 6 ft. 0 in. below the old foundations of the wall, the bottom of which is 3 ft. 0 in. below ground level. Specify suitable bricks and mortar for this work. Discuss critically methods of pinning up the new work to the old. I.O.B. 1939.

8. State the commercial grades of common clamp burnt bricks and give their suitable uses. Describe the characteristics of a good weather brick. I.O.B. 1939.

9. Draw to ½-inch scale sections of a fireplace opening in brickwork, showing the gathering over of the flue. Describe the operations of parging and coring. I.O.B. 1939.

10. Close timber planking 3 inches thick is to be used in the basement to afford additional protection in the event of an air raid. The planking is to be arranged as shown in Fig 1. Assuming a uniformly distributed load on the planking of 300 lbs. per square foot, calculate and draw the complete bending moment diagram for 1 foot of width of the planking and find the extreme fibre stress in the timber. Make the bending moment diagram to a scale of ½ inch to 1 foot.

Write a short explanation of the theory of beams of solid rectangular section such as timber beams, stating clearly how such beams act in resisting the shearing forces and bending moments and show how the formula for the resistance moment of a rectangular beam is arrived at. I.O.B. 1939.

11. Steel roof trusses of 40 foot span, 12 feet apart, are required to span between the walls of a single storey factory building. The slope of the roof is to be 1 in 2 and the covering is to be of asbestos-cement tiles laid on boarding supported by wooden purlins. The purlins are to be spaced at about 3 ft. 6 in. centre to centre up the slope. Draw a line diagram to a scale of ¼ inch to 1 foot of a suitable type of truss and

calculate the loads on the points of the truss, taking an inclusive load measured up the slope of 30 lb. per square foot of roof area all over. I.O.B. 1939.

Then find by graphic statics the forces in all the members of the truss, in tons, indicating compression by the plus sign and tension by the minus sign.

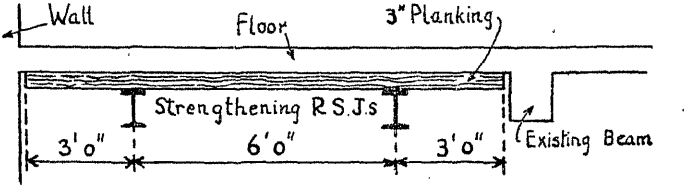


Fig. 1.

12. The floors of the building are to be composed of reinforced concrete, and consist of 14″ × 12″ main beams supported by a column and intersected with 6″ × 6″ secondary beams 1 ft. 6 in. apart on top of which is a 4-inch concrete slab.

To 1½-inch scale show how you would build up and strike the necessary form-work. Fully dimension your drawing. Show only sufficient to explain the intersection of the column mould with beam casing and the intersection of the secondary beams with main beams. I.O.B. 1939.

13. Some of the windows are to be fitted with linings which are splayed at an angle of 120° to the face of the frame. To 1½-inch scale draw a section, showing the window finishings. Also show how to obtain the bevel between the lining of the jambs and the linings at the head of the window. I.O.B. 1939.

14. The partitioning to form the Claims Superintendent's room is to be built up in hardwood, using 5″ × 3″ framing with 2-inch sashes in the upper part, the lower portion being fitted with 2-inch framing having raised and fielded panel with bolection mouldings both sides. To a 1-inch scale draw horizontal and vertical sections. Indicate how the bolection moulding is fixed, and to quarter full size draw a section through the angle post. I.O.B. 1939.

15. The main roof is finished with asphalt. Make detailed sketches showing :—

(*a*) The bossed portion of sheet lead necessary to convey water from the roof, through the wall and into a R.W. head.

(*b*) A 4-inch C.I. soil and vent pipe passing through the roof. Show the method adopted to make this pipe watertight to the asphalt. I.O.B. 1939.

16. The site plan shows the line of sewer and indicates the position of a 6-inch C.I. drain together with two 4-inch soil and vent pipes. Explain why a 6-inch pipe is laid to a fall of 1 in 60, and state why it is good practice to have a soil and vent pipe at either end of the drain.

Sketch :—

(*a*) 4-inch lead soil pipe connected to a C.I. drain bend.

(*b*) Joint made on the 6-inch C.I. drain. I.O.B. 1939.

17. The cold water storage cistern is to be placed upon the roof. What precautions must be taken when the cistern is fixed ?

What is the capacity in gallons if the cistern is 6 ft. 0 in. long, 4 ft. 0 in. wide and 2 ft. 6 in. up to the water line ?

What will be the pressure of water per square inch upon a tap 30 feet below the water line ? I.O.B. 1939.

18. Compare lead, cast iron and copper for use as waste and soil pipes. In what lengths can pipes in these metals be obtained, and how are they jointed and fixed ? I.O.B. 1939.

19. The outside of the building is to be finished in a factory mixed rendered finish. Specify a suitable material, and say how the work should be executed to avoid crazing. I.O.B. 1939.

20. Make a full-size drawing of a suitable section for the architrave moulding surrounding doorways and windows. The projection is to be $1\frac{3}{4}$ inches. Explain how this work is executed. I.O.B. 1939.

21. Write a full specification of materials and workmanship for the following plasterer's work :—

(*a*) Screed flat roof for asphalt.

(*b*) Screed lavatory walls for tiles. I.O.B. 1939.

22. The ceilings on the second floor are to be finished on plaster board. Write a specification for this work, and describe how it is carried out. I.O.B. 1939.

23. Describe the characteristics of a good quicklime for plastering finishing coat, its preparation for use and mistakes in its preparation to be avoided. Add notes upon the chemical composition of such limes and the deleterious substances which may be present liable to cause efflorescence. I.O.B. 1939.

24. The installation of low pressure hot water heating is suggested for the building. (*a*) By what method would you assess the amount of radiator surface required for each room ? (*b*) On what internal and external temperatures would your calculations be based ? (*c*) What allowance would you make for natural ventilation with shut windows and what effect would exposure have on the heat requirements ? I.O.B. 1939.

25. State the various conditions when the use of a mechanically driven accelerator is (*a*) essential, and (*b*) advisable in connection with a hot water heating installation. I.O.B. 1939.

26. (*a*) Calculate the water content of a hot water storage cylinder 2 ft. 0 in. diameter by 4 ft. 6 in. long.

(*b*) If this cylinder has been connected to a boiler which is too small, how would you reduce the effective cylinder capacity by, say, one third ? I.O.B. 1939.

27. Show, preferably by means of graphs, how the dimensions of a piece of timber are affected by changes in moisture content. Distinguish between the effects in the longitudinal radial, and tangential directions.

Give sketches, with *brief* notes, to show how the design of practical carpentry and joinery details is influenced by the phenomena described.

Mention a few examples which illustrate practical defects caused by lack of appreciation of the effects of moisture changes in timber. I.O.B. 1939.

28. A 14-inch brick wall is to be built in a moderately exposed situation. Considering only the durability and the weatherproof qualities of the wall, summarise concisely the relative advantages and disadvantages of different types of brick and mortar you might use.

Would you expect the nature of the "bond" to affect the weatherproof qualities of the wall ? I.O.B. 1939.

29. Give sketches of the following :—

(*a*) Section through the ridge of a roof covered with plain tiles, and provided with half round ridge tiles.

(*b*) Line diagrams of steel roof trusses suitable for slated roofs of 35 feet and 50 feet span. Distinguish between the tension and the compression members.

(*c*) Details of tubular scaffolding, showing how the forces involved are resisted. I.O.B. 1939.

30. Draw a section showing the foundations of an 18-inch wall.

(*a*) Show how you have determined the thickness of the concrete.

(*b*) The wall is 20 feet high above the concrete and carries a load of 8 tons per foot run in addition to its own weight. What stress per foot super is imposed upon the soil?

(*c*) Name two classes of soil which could safely be stressed to this extent and two which could not. I.O.B. 1940.

31. It is proposed to employ a 5-ton electric derrick crane with a 90 ft. 0 in. jib upon a building contract. The crane is to function 40 feet above ground level on three framed legs and staging.

Explain with sketches :—

(*a*) Why the leg supports of mast and back stays vary in construction.

(*b*) How you would apply the anchor and weight to one of the back stays. I.O.B. 1940.

32. It is proposed to provide an A.R.P. shelter in the basement of a building. The floor above the basement is not strong enough to carry the *debris* load which would fall upon it should the building collapse. As part of the strengthening scheme therefore it is proposed to provide central props under the main beams, these being simply supported at their ends. Explain how it is that such props strengthen the floor and illustrate your answer with sketches of relevant shearing force and bending moment diagrams. Calculations are not required. Are any special considerations to be borne in mind when dealing with reinforced concrete beams in this way? Sketch details of a tubular adjustable strut. I.O.B. 1940.

33. To $\frac{1}{4}$-inch scale, draw cross and longitudinal sections through the three-flue chimney stack : the first floor fire-place opening with relieving arch over to be detailed to $\frac{1}{2}$-inch scale. I.O.B. 1942.

34. Grade and describe the following :—

(i) Bituminous felts.
(ii) Insulating and building boards.
(iii) Blockboards.
(iv) Plasterboards.

Indicate by sketches some of the suitable uses of these materials. I.O.B. 1942.

35. Specify the constituents of :—

(i) Priming paint for wood.
(ii) Gloss paint.
(iii) Distemper.
(iv) Putty for—
(a) Wood.
(b) Metal. I.O.B. 1942.

36. (a) To ½-inch scale, draw an elevation and section of a timber framed centre for a semi-circular brick arch 10 ft. 0 in. span and 13½ in. thick. Name and dimension the parts of the centre.

(b) Sketch the following handrail details :—

(i) Joint with screw.
(ii) Ramp.
(iii) Wreath. I.O.B. 1942.

37. It is intended to form an opening 30′ 0″ long × 11′ 0″ high in the rear wall of an existing factory building, with new compound steel beam supported by stancheons at both ends to support the work over. The existing building is 60′ 0″ × 40′ 0″ on plan. It is two storeys high, and the floor to floor height for ground and first floors respectively is 14 ft. 0 in. and 10 ft. 0 in.

Detail the shoring necessary and explain briefly the method of executing all the work involved. I.O.B. 1942.

38. Describe the effects of frost on bricklaying. Discuss the risk of failure from this cause and the precautions which can be taken against it. I.O.B. 1942.

39. Discuss the composition and selection of mortars for brickwork. Give sketches of the various kinds of pointing and describe their execution. I.O.B. 1942.

40. Describe and illustrate with sketches the construction of drains and fittings under a building. I.O.B. 1942.

41. (a) The weathered pointed arch of a church window, formed of Darley Dale stone, shows the arch stones set bed way out.

How would you work and set the arch blocks of a similar opening ?

(b) Given your choice of Portland stone block for a heavy projecting cornice finished with dentil mouldings, how would you work, bed, and set the stone ?

I.O.B. 1942.

42. Describe the steps you would take in removing and replacing the defective arch referred to in question 41 to ensure the stability of the fabric during the process. I.O.B. 1942.

43. The R.S.J. has to carry a total uniformly distributed load of 14 tons. Calculate the maximum flexural and shear stresses if the joist is 12″ × 5″ with an inertia moment of 206 inches to the fourth and a web thickness of ·33 inch.

Also, in view of the need for steel economy, calculate an equivalent rectangular reinforced concrete beam 14 inches wide and 19 inches deep, and find the number and diameter of bars required as bottom reinforcement.

In both cases the span may be taken as 13 ft. 0 in. and freely supported. I.O.B. 1942.

44. A mass concrete A.R.P. wall 7 ft. 0 in. high and 2 ft. 6 in. thick has been built to resist blast pressure. If the blast pressure on the face of the wall is taken as equivalent to ½ lb. per square inch, uniformly distributed, calculate the maximum stresses in the wall (in lb. per square inch) taking into account the weight of the wall.

Describe and show by neat freehand sketches how you would build such a wall without the use of temporary shuttering. I.O.B. 1942.

45. For light-weight welded framing in wartime construction, channel-shaped pressed mild steel sections are being used. The outside measurements of the sections are 2″ × 1$\frac{1}{16}$″ and the thickness of the metal throughout is $\frac{1}{16}$ inch. Calculate the area and modulus of the section about the x–x axis and the radius of gyration about the axis y–y. Calculations to be taken to three places of decimals. I.O.B. 1942.

46. Write brief notes on the following :—

(*a*) Antisyphonage pipes.

(*b*) " A trap ventilating pipe shall . . . be connected with trap or the branch soil pipe or waste pipe (i) At a point not less than three or more than twelve inches from the highest part of the trap." (Drainage Bye-Laws.)

(*c*) Protective coatings for cast-iron pipes.

(*d*) The advantages of non-ferrous ventilating pipes.

I.O.B. 1942.

47. Make a sectional drawing of a cold water storage cistern to supply the building, incorporating the following features :—

Floor in reinforced concrete. Cistern in G.I. placed on bearers in a lead tray. Cistern sides and top covered to protect from cold and dust. Show positions of ball valve, distribution pipes and overflows from cistern and tray.

If the cistern has a capacity of 1,000 gallons and its dimensions on plan are 6′ 0″ × 5′ 0″, what height is attained by the water in the cistern ? I.O.B. 1942.

48. A chimney stack passes through the ridge of the building. The junction between the brickwork and the tiling is to be weathered with apron step flashing. Show how this is done. State poundage of lead used, and the purpose of the following :—

(*a*) Boxwood step turner.
(*b*) Lead bale clip or tack.
(*c*) Lead wedge.
(*d*) Water line. I.O.B. 1942.

49. The outside of the building is to be finished in plain stucco and jointed to imitate masonry. The coping and the cap to chimney stack are to be pre-cast. Write a brief specification for this work. I.O.B. 1942.

50. 400,000 B.Th.Units are required to be supplied to a building by a cast-iron sectional boiler in maintaining an inside temperature of 65° Fahr. when the outside temperature is at 32° Fahr. (*a*) How many heat units would be required to maintain an internal temperature of 65° Fahr. when the outer air is at 44° Fahr. ? (*b*) How much fuel would be used per week under each of the two outside conditions, assuming a boiler efficiency of 65 per cent. and a calorific value of 11,500 B.Th.Units per lb. for fuel ? For simplicity, the heat would be applied both day and night. I.O.B. 1942.

51. Briefly describe the composition, characteristics and uses of sand-lime bricks. C.S.I. 1938.

52. What are the chief points to be observed when selecting the following materials for general building purposes ?—

(*a*) Bricks for walling generally.
(*b*) Bricks for facing externally.
(*c*) Timber for carcassing.
(*d*) Timber for joinery.
(*e*) Sand for bricklaying mortar.
(*f*) Sand for internal plastering. C.S.I. 1938.

53. What are the three usual methods of seasoning timber ? Briefly comment on each. C.S.I. 1938.

54. Draw to a scale of 1 inch to 1 foot plans of two alternate courses of two 13½-inch walls in English bond meeting at a right-angle quoin. Mortar joints may be indicated by a single line. C.S.I. 1938.

55. Describe *briefly* the following terms relating to staircases :—

(*a*) Tread.
(*b*) Riser.
(*c*) Outer string.
(*d*) Newel.
(*e*) Spandril framing.

State any formulæ you know for determining the proportion between the " rise " and the " going." C.S.I. 1938.

56. A lead-covered roof is described as being " firred to a fall of 1½ inches in 10 feet." What does this mean ? Draw a section through a " drip " to a scale of half full size. C.S.I. 1938.

57. Draw to a scale of ¼ inch to 1 foot the plan of the joists of an upper-floor room in an ordinary domestic building. The room is 10′ × 15′ and has a fireplace opening 18 inches wide, with a 9-inch flue on each side, projecting 9 inches, midway in one of the longer sides. Show the trimming round the hearth and name and give the scantling of the timbers.

Quote any formulæ you know for determining the depth and thickness of the joists. C.S.I. 1938.

58. Describe *briefly* the following :—

(*a*) Soaker.
(*b*) P.T.G. flooring.
(*c*) Bressumer.
(*d*) Water bar.
(*e*) Apron piece.
(*f*) Knot, stop, prime and paint. C.S.I. 1938.

59. Describe the steps taken to form the wood lathing and plastering of a ceiling. Explain what is meant by " coarse stuff," " fine stuff," and " setting coat " and the constituents of each. C.S.I. 1938.

60. Draw to a scale of ½ inch to 1 foot an elevation and section of a three-light casement dormer window in the slope of a tiled roof. The dormer roof to be flat and covered with lead. The drawing to show the eaves, flooring of room and details in all trades. C.S.I. 1938.

61. Draw to a scale of ½ inch to 1 foot an elevation showing the construction of a timber-framed partition across a room 20 feet wide, with a doorway opening at each end 3 ft. 6 in. wide by 7 ft. 3 in. high. The partition to be constructed to support the floors. Height floor to floor 12 feet.

Half the elevation will suffice. C.S.I. 1938.

62. Draw to a scale of ½ inch to 1 foot an elevation of a queen post roof truss 40 ft. clear span, the ends resting on walls 2 ft. 3 in. thick.

Trusses are 10 feet apart. Figure the sizes and name the parts.

Half the elevation will suffice. C.S.I. 1938.

63. Draw to a scale of ½ inch to 1 foot a half plan elevation and section of an enclosed half-timbered, tile-roofed entrance porch to a small country house.

The half-timbered sides to be on a brick base 3 feet high—6′ × 6′ internal dimensions. Show a casement light window in each side, and gable end with barge board and entrance door.

The drawing to show details in all trades. C.S.I. 1938.

64. A parabolic three-pinned arch of 90 feet span and 12 feet rise has pin joints at crown and springings. It carries a uniformly distributed load of 130 tons over the whole span and 60 tons distributed uniformly from one end to the centre.

Find the horizontal thrust and maximum positive and negative bending moments.

65. A porch to the entrance door of a good class house is 4 ft. 6 in. square inside of 9 inch walls, and has a saddle-back roof of rafters, battens, felt and Broseley pattern tiles to a 3-inch lap and 45 degrees pitch, with a ceiling of ⅝-inch asbestos cement sheets at plate level. The boxed eaves and verge at front project 8 inch to face of fascia and barge boards respectively.

Draw to a scale of 1 inch to 1 foot a plan of the roof timbers and a cross section of the complete roof looking towards the face of the house, and indicate all the details for carpenter's, joiner's, tiler's, and plumber's work, including 4-inch half-round rainwater gutters and 3-inch dia. rainwater down pipes.

Indicate, by diagram or otherwise, the method of calculating the gauge for the tiling.

Dimension of all parts.

Mark where the following occur: Cover fillet, notching, scribing, splay-cutting, tilt-fillet.

Give weights per cubic foot of the following:—Baltic fir, oak (English), red pine, sheet lead, zinc. C.S.I. 1938.

66. Draw to a scale of 1 inch to 1 foot a plan, section and elevation of a two-light window in a 14-inch brick wall in English bond, size 3′ 6″ wide × 5′ 3″ high (clear opening) having :—

2-inch rebated and moulded casements of six panes each.
2-inch rebated and moulded fanlights of four panes each.
$4\frac{1}{2}''$ × 3″ frames, sill, mullion and transome.
$1\frac{1}{4}''$ × $\frac{1}{4}''$ galvanized water bar.
24 oz. clear sheet glass.
11″ × 6″ stone sill.
12-inch gauged brick arch and reinforced concrete lintel.
Inner quoins of opening to be bull-nosed in Keene's cement.
Dimension all the parts.

Show a small area of the face bone to brickwork about one reveal in elevation, and alternative courses in plan.

Mark where the following occur : Horn, scribing, skew-back, stooling, stub-tenon.

Give weights per cubic foot of the following : Mild steel, Portland stone (Whitbed), pressed brickwork, sheet glass, teak. C.S.I. 1938.

67. An oak entrance door with bronze furniture is set under a porch in straight jambs in a 15-inch limestone wall which has plastering on the inside. There is a 6-inch granite threshold to the opening, which is 7′ high × 3′ 3″ wide.

Draw to a scale of 1 inch to 1 foot a plan, section and elevation to show a 2-inch fully glazed door of $4\frac{1}{2}$-inch head and styles and 16-inch bottom rail, 1-inch bars in five lying panels, $4\frac{1}{2}''$ × 3″ frame with 1-inch linings and 4″ × $1\frac{1}{4}''$ architraves on wood grounds on each face.

Show alternate courses of bonding at each jamb.

Dimension all the parts.

Mark where the following occur : Dowel, rough ground, style, weathering, wrought ground.

Give weights per cubic foot of the following : Bronze, granite (Aberdeen), lime (fresh), limestone, oak (Baltic). C.S.I. 1938.

68. Draw to scale of 8 feet to 1 inch, ground floor plan of a factory building, length north to south 90 feet, width east to west 60 feet. The factory to be lit by north lights in the roof. Show positions of all internal stanchions and dot on plan the lines of supporting R.S.J.s and roof trusses. Figure on plan the sizes of the R.S.J.s and stanchions. Two doorway openings 10 feet wide by 10 feet high are required at the north and south ends of the building, fitted with roller shutters. The

height of the building from floor to top of stanchions is 13 feet. The enclosing walls may be of brickwork 9 inches thick, provided the weight of the R.S.J.s supporting the roof trusses is carried by stanchions built in the thickness of the walls.
C.S.I. 1938.

69. Draw to scale of 8 feet to 1 inch, a longitudinal section through the factory from north to south indicating the roof trusses and purlin positions to carry an asbestos-tile roof and north lights. C.S.I. 1938.

70. Calculate the weight on one of the internal stanchion bases, the total roof weight and superimposed load to be taken at $\frac{3}{4}$ cwt. per square foot of covered space. The ground is capable of safely supporting $1\frac{1}{2}$ tons per square foot. Give details of calculations showing how the figures are arrived at. State area of stanchion bases proposed. C.S.I. 1938.

71. Draw to inch scale plan and section of the stanchion bases above referred to and indicate the connection between the stanchion and the base. Figure on the size of the stanchion.
C.S.I. 1938.

72. Draw to inch scale a section through the eaves of the roof at the north end of the building, showing the eaves gutter size 6″ × $4\frac{1}{2}$″ cast iron and the finish of the feet of the roof trusses, rafters and purlins. C.S.I. 1938.

73. A house is to be demolished, situated in the centre of a row of houses. The houses on either side require shoring as the party walls are in an unsafe condition. The width between the houses is 16 feet, one party wall is 25 feet high, and the other party wall is 34 feet high. Both are to be supported from 1 foot above the ground level to within 4 feet of the top of the parapets. Draw to scale of 4 feet to 1 inch, an elevation of one bay of shoring you would suggest erecting and figure on the approximate sizes of the timbers. C.S.I. 1938.

74. Draw to one-quarter full-size scale :—

(*a*) Cross section through a 7″ × 4″ purlin fixed to a $3\frac{1}{2}$-inch wrot.-iron tee principal rafter and supporting 4″ × 2″ rafters and boarding ; the pitch of the roof being 30 degrees.

(*b*) Draw to same scale, section through the ridge of the same roof showing 9″ × $1\frac{1}{2}$″ deal ridge piece and 1-inch diameter wrot.-iron king tie. Show method of connecting all roof members at the ridge. C.S.I. 1938.

75. Draw to half full-size scale, section through the jamb of a double hung deal sash window in boxed frame. The frame is set back 4½ inches from the face of the wall. Figure on the dimensions of all members. C.S.I. 1938.

76. State briefly what you know about the "One Pipe" system of plumbing. In what class of buildings would you consider its adoption of advantage, and for what reasons? What precautions are necessary in connection with the wastes from sanitary fittings when this system is adopted? C.S.I. 1938.

77. Draw to scale of 4 feet to 1 inch, ground floor plan and front elevation of a brick porch to a private house. Dimensions 8 ft. 0 in. wide by 6 ft. 0 in. deep by 8 ft. 0 in. high, floor to ceiling. The walls are 1½ bricks thick—the floor is 1 ft. 0 in. above ground level. The porch is covered with a 6-inch thick concrete flat roof with 9 inches thick parapet wall over. Show double doors to inner and outer openings size 7 ft. 0 in. high by 4 ft. 6 in. wide, each door to be three panelled in oak and hung to 4½″ × 3″ frames. The two side lights are 2 ft. 0 in. wide. Show a brick on end course of bricks all round the porch at the level of door head. Sketch to quarter full size, section through door head showing how the brick on end course is supported across the opening. C.S.I. 1938.

78. A wall 1 ft. 10½ in. thick, built in English bond, carries 15 tons per foot of length (inclusive of wall footings and concrete base).

Calculate the width of concrete foundation, assuming the ground will safely carry 3 tons per square foot, and design the footings, concrete base and any reinforcement considered necessary.

Draw, to a scale of ½ in. to 1 ft., a plan of 5 feet run of the wall and a vertical section to show all the above, assuming the bottom of the concrete base to be 4 feet below ground line; the wall carried 1 ft. above ground line, and indicate position and type of damp-proof course.

Show all brick joints, reinforcement, etc., and indicate the method adopted to ascertain the depth of the footings, depth of concrete base, and the percentage of reinforcement in the latter.

Dimension all the parts.

Mark where the following occur:—Base, double course, header, offset, stretcher.

Give weight per cubic foot of the following:—Blue brickwork in cement, clay (damp), mild steel, Portland cement, reinforced concrete. C.S.I. 1938.

79. Draw to a scale of 1 inch to 1 foot a half cross section of a roof suitable for a small building 16 feet wide with 15½-inch cavity walls.

The roof is to be covered with ladies slates laid to a 3-inch lap on battens, counter-battens, felt and boards on rafters set at 45 degrees pitch, and finished with zinc roll ridge and boxed eaves projecting 8 inches from face of the wall.

The ceiling below is to be of "Tentest" with cover strips at 4-feet centres of the same material.

Show full details at the eaves, ridge, and ceiling and indicate any special hangers in the roof construction.

Dimension all members and parts, and state the formula used to arrive at the gauge for the slating when centre nailed.

Mark where the following occur: Beam filling, bed mould, hanger, half-lap joint, splay-cutting.

Give weights per cubic foot of the following: Pressed brickwork in cement mortar, snow (fresh), terra cotta (solid), Westmorland slates, zinc. C.S.I. 1938.

80. The ground floor of a cottage parlour, 12 feet wide by 10 feet 6 inches deep, is to be constructed of wood joists and flooring boards.

Two adjacent walls are external and built of 11-inch hollow brickwork, two others are half-brick partitions supporting an upper floor and abutting against solid floors.

A breast with fire opening 2 feet wide by 14 inches deep occurs at the external end, and opposite is a doorway 3 feet wide between brick jambs.

The floor level is 12 inches above the ground outside.

Draw to a scale of 1 inch to 1 foot a plan showing the carcasing, and a section showing the construction necessary to ensure through ventilation of the air space.

Dimension all members and show all joinery and plaster finishes.

Mark where the following occur: Air brick, damp-proof course, duct, fender wall, air space. C.S.I. 1938.

81. A rectangular bay window with $4\frac{1}{2}'' \times 3''$ solid frames and 2-inch casements measures $8' 9'' \times 1' 6''$ over the frames and has to be covered with a lead flat roof.

Draw to a scale of 1½ inches to 1 foot a half plan and a cross section.

The eaves project 6 inches beyond the face of the window frame and have 4 inches half-round rainwater gutters fixed on brackets.

Show on plan the wood bressumer supporting the 13½-inch main wall and all other constructional members for flat.

The cross section to include the head of the window frame and the bressumer over the inner opening with all lead and joinery work about the same.

The height of the window head is 7 feet above the floor level, whilst the room is 9 feet high.

Dimension all the members and parts.

Mark where the following occur:—Bearer, fall, fascia, fillet, lining. C.S.I. 1938.

82. A window opening in a stone wall 15 inches thick faced on both sides with squared or snecked rubble masonry measures 2 ft. 7 in. wide by 5 feet between a 17″ × 6″ stone sill and a 15″ × 9″ stone head.

Draw the following to a scale of 3 inches to 1 foot:—

(*a*) Half-plan with a reveal 6 inches deep and a 2-inch recess to show 5″ × 4″ frame (exposed 3 inches on the outside) with one 4″ × 2″ centre glazing bar and beads and ¼-inch plate glass with all finishings.

(*b*) Complete section (which may be broken in height to save space) to show the stone sill and head, and 5″ × 4″ wood sill, head and transome with one 4″ × 2″ glazing bar, glass and all finishings.

The window frame is to be glazed direct with beads.

Dimension all the members and parts.

Show and describe a method of securing the frame into the recess of wall.

Mark where the following occur: Mastic pointing, pinning, scribing, stool, sub-tenon. C.S.I. 1938.

83. Show by suitable diagrams the following systems of heating three-storey buildings by hot-water:—

(*a*) One-pipe drop system.

(*b*) Drop system for panel heating. C.S.I. 1938.

84. Explain the meaning of the expression "heat losses" in connection with the heating of buildings. State briefly the allowances which have to be made for this factor when designing a heating system. How can heat losses be minimized by the methods of the construction of buildings? C.S.I. 1938.

85. Define:—

(*a*) British Thermal Unit.

(*b*) Therm.

(*c*) The Calorie.

(*d*) Specific heat.

(*e*) Latent heat. C.S.I. 1938.

86. Describe :—

(*a*) The Anemometer.
(*b*) Pitot Tube.
(*c*) Kata Thermometer.

How are these instruments used in ventilation problems ?
C.S.I. 1938.

87. Explain the need to warm the air admitted into buildings by ventilation systems. Make a sketch section of a small factory with machines which create dust and indicate thereon a suitable scheme of ventilation on the extraction system.
C.S.I. 1938.

88. What are ?—

(*a*) Ventilation standards.
(*b*) Air conditioning.

How are ventilation standards affected by air-conditioning plants ? C.S.I. 1938.

89. On inspecting the exterior of a house, you observe a stack of iron soil pipe receiving branches from two W.C.'s on upper floors ; it is in a position which you cannot get near, and is roughly 10 feet away from you. Explain the details of its appearance which would guide you in deciding whether it was of proper heavy soil piping or merely of rainwater piping.
C.S.I. 1938.

90. A country residence, in an isolated position, has a small private sewage disposal plant comprising a liquefying tank (6 ft. 6 in. long by 1 ft. 6 in. wide and 5 ft. 6 in. deep to water level) and a filter bed (10 ft. long by 5 ft. wide by 5 ft. high) fully exposed to the air on three sides. Explain briefly :—

(*a*) What happens to solid fæces which are discharged into the tank (if necessary, draw a diagram to explain your answer).
(*b*) To what extent the sewage is altered by its passage through the tank.
(*c*) Whether the term " filter bed " accurately conveys the true meaning of its function ; if not, in what way is such term a misnomer.
(*d*) What really happens to the effluent during its passage through the filter bed.
(*e*) What would be the effect of passing the sewage direct on to the filter bed without having previously passed it through the tank. C.S.I. 1938.

91. A factory, which has occasion to use rainwater in its process, has a large rainwater reservoir in its basement which will hold 35,000 gallons and into which discharges all the rainwater from an impervious area of 38,400 square feet; the overflow from that reservoir is a 4-inch pipe running underground at a gradient of 1 in 40 to a dyke some 100 yards away.

Will this pipe be able to deal with a steady rainfall of a quarter-inch per hour for two consecutive hours if it should occur when the reservoir is full at the commencement and it happens at night when the works are shut down?

Hydraulic data for 4-inch pipe running full is as follows:—

Sectional area of flow in square feet ..	·087266
Hydraulic mean depth in feet ..	·083333

C.S.I. 1938.

92. (*a*) What do you understand by the term "moment"? Give illustrations of its use in structural calculations.

(*b*) Explain the difference between section modulus and moment of resistance.

(*c*) A rod AE 8 ft. 0 in. long weighing ½ lb. per foot run rests on bearings at B, 2 ft. 0 in. from A, and at E, 6 ft. 0 in. from B. At C, 4 ft. 0 in. from A, it carries a load of 2 lbs., and at D, 6 ft. 0 in. from A, a load of 4 lbs. What load at A must be exceeded in order to cause the end E to rise?

(*d*) The handle of a door 3 ft. 0 in. wide by 6 ft. 8 in. high is situated at 2 ft. 8 in. from the hinge. What horizontal wind pressure per square foot would prevent the door from being opened by a horizontal force of 45 lbs. on the handle? What force exerted in a direction inclined at 45 degrees with the plane of the door would be required to overcome that wind pressure?

(*e*) Explain why and how the relative stiffness of steel and concrete, as measured by their respective moduli of elasticity, enters into calculations for reinforced concrete. C.S.I. 1938.

93. A girder AD 12 ft. 0 in. span carries a distributed load, including its own weight, of ½ cwt. per foot run. At B, 4 ft. 0 in. from A, it carries a concentrated load of 8 cwts., and at C, 8 ft. 0 in. from A, a concentrated load of 4 cwts.

(*a*) Calculate the reactions at A and D.

(*b*) Draw the shear diagram.

(*c*) Locate the maximum bending moment.

(*d*) Calculate its amount in foot cwts.
(*e*) Specify the dimensions of a rectangular beam section on which the extreme fibre stress in bending should not exceed 12 cwts. per square inch. C.S.I. 1938.

94. Let the girder AD in Question 93 be extended over the bearing D to E, 4 ft. 0 in. from D, as a cantilever carrying an equally distributed load including its own weight of $\frac{1}{2}$ cwt. per foot run, and at E a concentrated load of 28 cwts.

(*a*) Calculate the reactions at A and D.
(*b*) Draw the shear diagram.
(*c*) Locate the position of the maximum bending moment.
(*d*) Calculate its amount in foot cwts. and sketch the bending moment diagram.
(*e*) Specify the dimensions of a rectangular beam section in which the extreme fibre stress should not exceed 12 cwts. per square inch. C.S.I. 1938.

95. State any rules with which you are familiar relating to the design of riveted joints.

If the permissible stresses on mild steel in tons per square inch be 5·5 in single shear, 9·625 in double shear and 11 in bearing, what load can be placed on one rivet $\frac{3}{4}$ inch diameter in $\frac{1}{2}$ inch plate ?—

(*a*) In single shear.
(*b*) In double shear. C.S.I. 1938.

96. Draw to a scale of not less than $\frac{1}{2}$ inch to 1 foot, or give fully dimensioned sketches, of a riveted stanchion 14 ft. 0 in. long overall consisting of a 12″ × 6″ rolled joist with 10″ × $\frac{1}{2}$″ plates riveted on each side carrying an axial load, including its own weight, of 120 tons. Rivets $\frac{3}{4}$ inch diameter to be used throughout. Cap plate 18″ × 12″ × $\frac{1}{2}$″. Base to have sufficient rivets to transmit load to base plate 2′ 6″ × 2′ 6″ × $\frac{5}{8}$″.

Permissible stresses on steel as in Question 95.

Foundations granite base on mass concrete.

Permissible loads per square foot 12 tons on concrete and 4 tons on ground. Weight of foundations may be neglected. C.S.I. 1938.

97. Calculate longitudinal reinforcement for an alternative design of the column in Question 96 in reinforced concrete 14″ × 14″ overall. Appropriate minimum hoop reinforcement may be assumed.

Give dimensioned sketch of base in reinforced concrete to distribute load on ground at 4 tons per square foot. Weight of column and base may be neglected. C.S.I. 1938.

98. A reinforced concrete cantilever 2 ft. 0 in. long, 12 in. wide and 16⅜ in. effective depth carries an equally distributed load, including its own weight, of 8 tons ; is composed of steel and concrete upon which the permissible working stresses per square inch are 16,000 lbs. and 600 lbs. respectively.

(*a*) Specify tensile reinforcement which would comply with an economic ratio of 0·675 per cent.

For the tensile reinforcement selected calculate :—

(*b*) The position of the neutral axis from the formula $n_1 = \sqrt{(m^2r^2 + 2mr} - mr)$ $(m = 15)$.

(*c*) The lever arm.

(*d*) The stress on the concrete.

(*e*) The stress on the steel.

(*f*) Explain why in this case the economic ratio is extravagant. C.S.I. 1938.

99. What do you understand by the expression " underpinning " ? Describe the steps necessary to underpin a wall which shows evidence of settlement extending down to the footings. C.S.I. 1939.

100. Draw to a scale of 1½ inch to 1 foot a cross section on Line C—D at roof level to show the 6-inch stone coping, 9-inch brick parapet (rendered at back in cement mortar), a reinforced concrete lintel for full thickness of wall to door opening, and a 9-inch soldier arch with inside concrete lintel to the louvred ventilator opening ; also the thickness and finishes of flat roof.

Indicate, by a section, an outlet for rain water from the roof surface into a rain water head.

Dimension all the parts, and state the total safe load (superimposed) per square foot which may be allowed on the flat roof for asphalt, snow and traffic for repair.

Mark where the following occur : Cement rendering, damp-proof course, drip, fall, throating.

Give weights per cubic foot of the following : Bath stone, lead (sheet), natural rock asphalt, reinforced concrete, steel. C.S.I. 1939.

101. Draw to a scale of 1½ inch to 1 foot a plan, section and an elevation of the louvred ventilator for the opening lettered E, being 1 ft. 8 in. wide and 2 ft. 9 in. high from top of 6-inch stone sill to underside of a 9-inch arch outside, and having an oak sill 4½″ × 3″, frame 4½″ × 3″, with 6″ × 1″ oak louvres at 3-inch pitch and 30 degrees rake.

Indicate the method to be employed to prevent moisture in the cavity of wall from running down on to the frame head, and its ultimate disposal ; alternate courses of brick bonding at

the jambs and reveals, and to one-half of the elevation ; show ties. Both faces of wall are pointed and the internal sill is of bull-nosed bricks.

Dimension all the parts, and—

Mark where the following occur : Cover mould, stooling, throating, water bar, weathering.

Give weights per cubic foot of the following : Brass (cast), glass (sheet), oak (English), teak (Burma), pitch. C.S.I. 1939.

102. Draw, to the scale stated, any TWO of the following :— Dimension all members and parts in detail.

(*a*) Isometric projection to half full size of a mortice and tenon joint with top and bottom shoulders on a 5″ × 2″ scantling.

(*b*) Isometric projection to half full size of a dovetailed halved joint when two 4″ × 3″ wall plates meet at right angles (Show each member separated.)

(*c*) A half section and a half elevation to full size of a flanged joint where a 2-inch diameter lead pipe passes through a 1¼-inch wood floor. Indicate the direction of flow.

(*d*) A half section and a half elevation to half full size of the joint between the end of the outlet from a W.C. trap and a lead soil pipe. C.S.I. 1939.

103. A level site with 150 feet frontage and depth of 250 feet on the north side of an Arterial Road running east to west has been purchased for the erection of a petrol filling station and small repair garage with the intention of extending the latter at a later date.

The filling station is to comprise six pumps and the other usual amenities on an island protected from the weather by a glazed steel canopy cantilevered out from a building containing :—

Showroom about 20′ × 15′ with showcases.
Cashier's cabin and clerk's office.
Manager's office.
Staff room and lavatory (men only).
Lavatories for patrons (both sexes).

Attached to and behind this building is to be the first bay of the repair shop which is to be 60 feet long (east to west) and 40 feet wide, with entrance at west end. Work benches and small machine tools are to be located in this building. An inspection pit is to be provided and store room for small parts, tools and tyres (the latter may be accommodated in an overhead gallery if desired), staff mess room and lavatory.

The general construction to be of brick with steel trusses and stanchions where necessary to make the north wall of a temporary nature. The roof of the repair shop in rear is to be at 1/3 pitch with corrugated asbestos sheeting and patent glazing, the height being 12 feet to truss tie. Steel trusses should be strengthened to bear lifting gear up to additional load of ½ ton. The south elevation to the road is to be of advertisement value.

The method of roofing the front portion of the garage is left to the discretion of the candidate. C.S.I. 1939.

104. State what unit loads, for the various materials employed, you allow in calculating the pressure on the foundations of the stanchions in the north wall. Detail one typical base foundation to 1-inch scale and show that a load of 1·5 tons per square foot on the subsoil is not exceeded.

105. (*a*) Draw to ½-inch scale details of inspection pit showing arrangements for draining, etc.

(*b*) What steps do you take as regards waterproofing the pit in a very wet subsoil? Write your reply on the drawing. C.S.I. 1939.

106. Draw a typical ½-inch scale detail section through an external wall of a two-storey building with 11-inch brick cavity walls, boarded floors on 9″ × 3″ timber joists (showing bearings), timber casement windows total width 6 feet to first floor and glazed double folding doors to ground floor, both openings having brick on end arches over. Tiled roof behind parapet and hipped dormer window to attic are to be shown. Dimension and name all members. C.S.I. 1939.

107. Draw half F.S. details of a suitable pair of timber casement windows (opening out) for an exposed locality; casements 4 feet high and without centre mullion. Schedule the ironmongery and furniture required. C.S.I. 1939.

108. An opening 15 feet wide is to be formed at ground floor level near to one corner of an old two-storey non-basement house and a three-sided timber framed bay window with flat copper roof erected. Metal sliding-folding window on one side and fixed windows on the other two sides are required. Assume floor to floor heights of 10 feet and that the first floor joists at present bear on the external wall in question; projection of bay window to be 4 feet from the main wall face.

(*a*) Draw to ¼-inch scale your scheme for ensuring the safety of the existing structure during the alteration and state approximate sizes of the shores, etc., used.
(*b*) Draw to ½-inch scale plans and elevations of the new bay window.
(*c*) Draw to 1-inch scale cross-section through the bay to show the general construction with such details as would be required by a builder clearly indicated.

C.S.I. 1939.

109. Draw to a scale of 1½ inches to 1 foot two cross-sections through a flat roof with provision for a lantern light. The roof is to be formed with 7″ × 4″ R.S.J.'s about the opening and 4″ × 1¾″ steel filler joists at 2 feet 0 inch centres. The top flanges are to be level, and the main girders to be cased in concrete. The sections are to be taken to 9 inches below the flat and to show a 1 ft. 6 in. parapet above the roof surface. Omit the roof light.

Indicate the thickness and finishes of the flat roof and manner of making a watertight joint against the parapet walls.

Dimension all the members and parts, and state what formula should be used to arrive at the size of the main girder and filler joists, allowing 1½ cwt. per square foot of surface for inclusive loading.

Mark where the following occur: Cove, damp-proof course, flange, weathering, web.

Give weights per cubic foot of the following: Lime plaster, natural rock asphalt, reinforced concrete, steel, York stone.

110. Draw to a scale of 1½ inches to 1 foot a reinforced concrete raft foundation. The raft to extend 6 inches beyond the whole area of the building and its surface to be 12 inches below the ground level.

Indicate alternative methods of dealing with the base of the 11-inch cavity walls, also methods of rendering the walls and floor watertight, and to prevent damp from rising up the walls. The floor (12 inches above ground level) is to be of 1-inch boards on 4″ × 3″ joists and plates with the usual ventilated space below.

Dimension all the parts and members.

The cross-section may be broken, but must indicate an intermediate method of support to the wood joists.

Mark where the following occur: Cranked bar, ventilated space, offset, plate, reinforcement.

Give weights per cubic foot of the following: Clay, damp, plastic. Common brickwork in lime mortar. Reinforced concrete, slate, water (fresh).

111. Draw to a scale of 1½ inches to 1 foot a half plan through the curb, a half plan at top, and a cross-section having a concrete curb 4½″ × 6″ high above the 6-inch concrete flat roof, and vertical walls of 9″ × 9″ × 4½″ louvred air bricks along the 3-foot sides and 4½-inch solid brickwork along the 2 ft. 6 in. sides, and the top of 3 in. thick Glascrete construction. (Lenses being 7″ × 7″ × 1″).

Indicate method of finishing asphalt of roof covering about the light.

Dimension all members and parts.

Mark where the following occur: Angle fillet, bed joint, groove (or raglet), perpend, throating.

112. Draw to a scale of 1 inch to 1 foot a plan (throughwebs) and two cross-sectional elevations to show the jointing between the main girder (7″ × 4″) and the steel filler joists (4″ × 1¾″) in the flat roof at their intersection, when the tops of each are kept level.

Differentiate between rivets and bolts by a schedule of references, and indicate where each would be used in making the connections.

Dimension all the members and parts.

Mark where the following occur: Bearer bracket, connecting bracket, heel, toe, web.

113. You are consulted by a client with reference to periodical flooding which occurs in a cellar of his house. He tells you that there is a gully, connected to the drain in this cellar and that occasionally; when there is a heavy fall of rain, water rises through this gully and floods the cellar, disappearing when the rain ceases. He does not wish the drain to be sealed up as the cellar has to be constantly swilled.

In what way would you advise him to deal with the trouble? By means of a sketch, illustrate what you propose. C.S.I. 1939.

114. What vertical pipes would you expect to find on the outside of an ordinary dwelling-house? Group these according to the purposes for which they are used and indicate the manner in which the tops and bottoms of these pipes are dealt with. How would these pipes be fixed to the wall? C.S.I. 1939.

115. Illustrate by means of sketches :—

(*a*) An automatic flushing tank suitable for use in connection with a range of urinal stalls, and

(*b*) Some form of water waste preventing cistern with which you are acquainted.

Give the water content of each and explain how syphonage is set up in each case. C.S.I. 1939.

116. How and why are house drains ventilated ? Illustrate your answer by means of a diagrammatic sketch and mention any particular " sizing " of pipes, etc., which would engage your attention when designing a drainage system. C.S.I. 1939.

117. You are asked to arrange for a new sink to be fixed in the scullery of a small house. What type would you recommend ? State the approximate sizes and the height of the top of the sink from the floor, when fixed. How would the waste be dealt with ? C.S.I. 1939.

118. Make suitable sketches of the following hot water heating systems : (*a*) Two pipe up feed ; (*b*) Two pipe drop. C.S.I. 1939.

119. What is a calorifier or indirect heater ? Make a sketch diagram of a hot water supply system with a storage calorifier heated by steam. C.S.I. 1939.

120. Make a sketch cross section of a two-storey building with a basement and indicate a method of extract ventilation. C.S.I. 1939.

121. Explain the advantages of air conditioning. Make a sketch plan showing the layout of an air conditioning plant for a public building. C.S.I. 1939.

122. Give the requirements of the model building bye-laws with respect to the ventilation of rooms. Make sketches of some appliances used in natural ventilation. C.S.I. 1939.

123. (*a*) Explain in simple terms what you understand by the difference between bending moment, section modulus and moment of resistance and express the relation between them.

(*b*) A load of 30 cwt. is hanging on a wire rope 25 feet long. It has to be pulled aside 20 feet in a horizontal direction and is then 15 feet vertically below the point of support. What is then the tension in the rope ?,

(*c*) A beam AD 20 feet long carrying an equally distributed load including its own weight of 10 lb. per foot run rests on two supports B and C 2 feet and 12 feet respectively from A. At A it carries a concentrated load of 400 lb. and at B a concentrated load of 600 lb.
(1) Is it stable ?
(2) What are the vertical reactions at B and C ?

(*d*) Define the terms :—
(1) Modulus of elasticity.
(2) Modular ratio.
What values are ascribed to them in steel and 1 : 2 : 4 concrete ?

(*e*) In a rectangular concrete beam section 6 inches wide and 11 inches deep, reinforced in tension only, the centre of the reinforcement is 1 inch from the bottom edge of the section and the neutral axis (n) 3·6 inches from the top edge.
(1) What is the length of the lever arm (a) ?
(2) If the area in cross-section of the tensile reinforcement is 0·405 square inch and the safe stresses on the steel and concrete 16,000 and 600 lb. per square inch respectively, what is the maximum bending moment which can be applied to the section ? C.S.I. 1939.

124. (*a*) What are the values of n_1, a_1 and r of the beam section specified in Question 1 (*e*) ?
(*b*) To what extent would the formula $M = 95bd^2$ be applicable to this section ?
(*c*) State the limitations in the permissible application of that formula. C.S.I. 1939.

125. What are the bending moments at B and C on the beam specified in Question 123 (*c*) ?
Draw the shear diagram.
What is the nature (*i.e.*, whether hogging or sagging) and amount of the bending moment midway between B and C ?
C.S.I. 1939.

126. A rolled joist section 6″ × 14″ with a web 0·4 inch and flanges average 0·7 inch thick is available but no tables of strengths.
Calculate its approximate moments of inertia, section moduli and radii of gyration—
(*a*) In a direction parallel with the web ;
(*b*) In a direction at right angles to the web. C.S.I. 1939.

127. It is proposed to use the rolled joist specified in Question 126 over a span AD of 16 feet to carry an equally distributed load, including its own weight, of ½ ton per foot run and two concentrated loads B and C of 4 tons each at points respectively 4 feet and 10 feet from A.

(*a*) Calculate the reactions at A and D.
(*b*) Draw the shear diagram.
(*c*) Determine by calculation the position of the maximum bending moment.
(*d*) Calculate the amount of the maximum bending moment in inch-tons.
(*e*) From your calculations of the approximate section modulus determine the maximum stress in flexure.

C.S.I. 1939.

128. Design vertical reinforcement for circular reinforced concrete column 9 inches external diameter over cover and 9 feet effective length to carry load at bearing A of the beam specified in Question 127. Permissible load 60 per cent. of permissible load on short column. Permissible stress on concrete 600 lb. per square inch, modular ratio 15. Assume minimum lateral reinforcement. C.S.I. 1939.

129. A riveted steel roof truss 30 feet span 30 degrees pitch with horizontal tie carries ridge and two purlins on each slope, each imposing on the truss a load of 30 cwts., which may be assumed to include weight of purlins, ridge and truss.

Draw skeleton truss. Determine nature and amount of stress in each member. Specify suitable sections. Calculate rivets required and detail joints. C.S.I. 1939.

130. Write a specification for a best quality hand-made, sand-faced, red facing brick, giving its properties and characteristics and the tests that it should be required to pass.

C.S.I. 1940.

131. Set out the principal differences between Portland cement and rapid-hardening cement, both as regards their composition and their use. C.S.I. 1940.

132. Give one reason why Gypsum plasters, such as Keenes cement and Sirapite plaster, are unsuitable for use in external work and state two examples of where you would recommend the use of Keenes cement for internal work. C.S.I. 1940.

133. Describe briefly the difference between woods classed commercially as "hardwoods" and "softwoods" and give three examples of each classification. C.S.I. 1940.

134. (*a*) What are the chief constituents of glass ?
(*b*) Describe the processes in the manufacture of sheet glass and plate glass. C.S.I. 1940.

135. Briefly define the following terms :—
(*a*) " Oolitic " in relation to limestones.
(*b*) " Wainscot " in relation to oak.
(*c*) " Medullar Rays " and " Waney Edges " in relation to timber. C.S.I. 1940.

136. Draw to a scale of 1 inch to 1 foot, the plan and internal elevation of a 2¼-inch skeleton framed, ledged and braced door, filled in with ⅞-inch matched and " V " jointed boarding and hung to a 4½″ × 3″ rebated frame. Indicate on which side the door is hung. Size of door 2′ 8″ × 6′ 8″. The wall in which it occurs is a 9-inch external wall plastered on inside face. C.S.I 1940.

137. (*a*) Draw to a scale of 1½ inch to 1 foot, a section through the tusk tenon joint between a 9″ × 3″ trimming joist and a 9″ × 3″ trimmer.
(*b*) State briefly, illustrating by sketch if desired, the main reason for its peculiarity of shape. C.S.I. 1940.

138. Draw to a scale of 1 inch to 1 foot, the elevation and section of the joint between a king post and tie beam :—

King post out of 7″ × 6″.
Tie beam 11″ × 6″.
Struts 6″× 6″. C.S.I. 1940.

139. Describe the following (with sketches if desired), and state their respective purposes :—
(*a*) Soldered dot.
(*b*) Rag bolt.
(*c*) Coping iron.
(*d*) Template.
(*e*) Coping stone.
(*f*) Lead soaker. C.S.I. 1940.

140. Draw to a scale of 1 inch to 1 foot the plans of two alternate courses of the bonding at the junction of an 18-inch external wall and a 13½-inch cross wall, both built in English bond. Joints may be indicated with single line.
C.S.I. 1940.

141. What precaution should be taken to avoid cracking of wall plastering due to expansion and contraction of heating mains where passing through walls ? C.S.I. 1940.

142. State reason for which sleeper walls under ground floor joists are built honeycomb. C.S.I. 1940.

143. Describe the following (with sketches if desired), and state their respective purposes :—

(*a*) Footings.
(*b*) Dowel pin.
(*c*) Dog.
(*d*) Throating.
(*e*) Herring-bone strutting.
(*f*) Lead tack or tingle. C.S.I. 1940.

144. When fletton brickwork is required to be plastered what steps should be taken to obtain a good key for the plaster ? C.S.I. 1940.

145. Draw to a scale of 1 inch to 1 foot, a section through the top of an 11-inch cavity wall showing beam filling, roof plate, 4″ × 2″ rafters at 45 degrees pitch, fascia, soffit boarding, gutter, etc., and a few courses of plain roof tiling laid to 4-inch gauge. C.S.I. 1940.

146. What is meant by " knot, stop and prime " ? Give the constituents of each. C.S.I. 1940.

147. Draw to one-quarter full size, plan through one jamb of a door opening in 9-inch wall, show rebated linings, architrave, 2-inch door bolection moulded one side and planted moulding on reverse side. C.S.I. 1940.

148. Draw to a scale of ½ inch to 1 foot a section through the upper part of a building 18 feet wide external dimensions roofed with a mansard roof. The section to be taken through a dormer window in front having a lead flat roof, and to show the joists, skirting, eaves gutters and the room in the roof complete. C.S.I. 1940.

149. Draw to a scale of ½ inch to 1 foot a plan and section of an open newel staircase with a quarter space landing, from the ground floor to the first floor of a dwelling house. Height from floor to floor 10 ft. 6 in. Clear width between walls, 7 feet. C.S.I. 1940.

150. Draw to a scale of ½ inch to 1 foot a half plan, half elevation, and section of a vestibule screen 12 feet wide by 11 feet high, with a pair of swing doors in the centre with side light and fanlight over. C.S.I. 1940.

151. Draw to a scale of ½ inch to 1 foot an elevation of a flying shore between two buildings 20 feet apart.
Name the parts and figure the sizes. C.S.I. 1940.

152. Complete with first floor plan with roof plan dotted, cross section, front and back elevations, the eighth scale drawings for detached house in semi-rural situation, with clay sub-soil, and sewer in roadway at front. A start has been made upon the sheet with a partly completed ground floor plan. The external walls to be brick faced, and the whole of the construction to be that which you would recommend for a thoroughly sound modern job. The accommodation of first floor should be at least four bedrooms, bathroom, W.C., linen cupboard, etc. Complete the ground floor plan with drainage, inspection chambers, etc. C.S.I. 1940.

153. Indicate with note on plan how some portion of the construction could be so varied as to provide air raid shelter accommodation within the premises. C.S.I. 1940.

154. Complete your sheet with the following small marginal half-inch details :—

(*a*) Section of foundations to external walls with ground floor construction recommended.

(*b*) Section of external wallhead and roof construction.

C.S.I. 1940.

155. Draw a section to half-inch scale through the fascia and windows over a shop which has two storeys above it. Roller shutters, which may be accommodated in bulkhead seat under first floor window, are required, and the fascia is to be so arranged that the bottom is level with the ceiling. The front wall to first and second floors, which is 11 inches thick, is to be carried up with high parapet surmounted by artificial stone coping. The floors and flat roof are of hollow tile and concrete construction, the former covered with cork flooring and the latter with asphalt. The window heads are to be formed with flat brick arches externally, and with reinforced concrete lintols behind as constituent part of floor over. The windows to first floor to be double hung sashes, and those to second floor steel casements direct to brickwork. All heights of windows may be broken. Every precaution should be taken to resist the penetration of dampness. C.S.I. 1940.

156. Draw to inch scale, plan of reveal of second floor window. C.S.I. 1940.

157. When making a survey of such a building single handed, how would you arrive at the party wall thickness at each floor level ? Illustrate with freehand sketch. C.S.I. 1940.

158. State briefly the difference between cement and lime

as constituent parts of mortar. Make freehand sketch sections of various types of pointing suitable for each.

C.S.I. 1940.

159. Draw to a scale of 1 inch to 1 foot, two half-plans (one above the leadwork and the other to show the timber framing), a half-cross section, and a longitudinal section of a lead-covered flat roof over a space measuring 8′ × 6′ and having three 9-inch brick walls below the roof and the fourth 9-inch wall (on left-hand side) carried 1 foot above the roof level to top of a 11″ × 6″ stone coping.

The ceiling to space below is to be of plaster on expanded metal lathing.

Show all joists (to run across the shortest span), trimmers, drips, fillets, boarding, and a boxed eaves with 1 inch fascia and soffite boards and a 2½″ × 1½″ bed mould; all to project 6 inches from face of three walls; also all leadwork and a means of discharging the rain water from the roof.

Dimension all the members and parts, and indicate any special precaution taken to ventilate the spaces enclosed by the roof framings.

Mark in red pencil where the following occur :—

Angle fillet, bed mould, cover flashing, drip, stopped end.

160. Draw to a scale of 1½ inches to 1 foot the plan, section and half elevations of exterior and interior of a framed, ledged and braced door measuring 2 ft. 9 in. wide by 6 ft. 9 in. high, hung to open inwards in a 4½″ × 3″ rebated and moulded frame set in 4½-inch outer reveals and 2¼-inch recesses in a 13½-inch brick wall having a flat gauged brick arch 4 courses deep on outside and a reinforced concrete lintel 2 courses deep on inside. The opening to be finished with 1⅛ inch bull noses to reveals outside and linings and architraves, etc., for plastered walls on inside.

Indicate English bond for alternate courses in plan and elevation and all the finishings to both elevations; also the manner of ascertaining the size of brick and setting out for the gauged arch, and any special precautions to prevent rain from lodging on the front of the door itself or from gaining access to the inner wood floor.

Dimension all the members and parts and show how the frame is secured into the brickwork at step, jambs and head.

Show by dotted lines all the joints between braces, rails and styles of door.

Mark in red pencil where the following occur :—

Bare-faced tenon, dowel, horn, king closer, queen closer.

Give weights per cubic foot to the following :—

Asphalt	lbs.....
Columbian pine	lbs.....
Pressed brickwork in cement mortar ...	lbs.....
York stone	lbs.....
Wrought iron	lbs.....

C.S.I. 1940.

161. Draw to a scale of 3 inches to 1 foot the plan, section and half elevations of exterior and interior of a folding casement window (without mullions) made to open outwards in a wood framed building constructed of 4″ × 2″ studs and noggings. The exterior to be covered with feather-edged and rebated weather-boarding ex. 5″ × 1″ and the interior with ¾ inch T and G and channelled matched boarding, both set horizontally.

The window is to be constructed of 5½″ × 2″ and 7″ × 3″ double sunk and splayed and grooved on top and throated on underside oak sill, 2-inch sashes having 2″ × 2½″ rebated meeting styles and 2″ × 2½″ splayed, rebated and grooved bottom rail and two 1 inch rebated horizontal glazing bars in each sash; and finished inside and outside with 3″ × 1″ plain architraves, and 24 oz. clear sheet glass in putty.

The frame is to measure 2 ft. 5 in. wide by 2 ft. high in clear opening between sides, sill and head respectively.

Dimension all the members and parts, and indicate in red pencil special precautions taken to make a watertight joint at the top and bottom of the window frame.

Mark in red pencil where the following occur :—

Head, mitred joint, scribed joint, stud, throating.

Give weights per cubic foot to the following :—

Cedar	lbs.....
Copper	lbs.....
Glass (sheet)	lbs.....
Oak (English)	lbs.....
Zinc	lbs.....

C.S.I. 1940.

162. Draw to a scale of 1 inch to 1 foot the plan and cross section of a lift well to measure 6′ × 6′ between trimmers in a wood floor constructed with 11″ × 2″ joists, 1¼ inch T and G flooring, with expanded metal lathing to plaster ceiling below. There are no walls about the well hole which is in the centre of a square room.

One-half of plan and cross section are to show the carcasing and the other half of each the finishings.

Indicate the run of floor boards, bridging, trimmed, trimmer and trimming joists with all joists.

Draw to half full size a sectional elevation of the joint between the trimmer and trimming joists.

Draw to half full size a section at edge of the well hole to show method of finishing to floor, ceiling and side of well (there will be a guard rail—not to be indicated).

Dimension all the members and parts.

Mark in red pencil where the following occur :—

Cross tongue, horn, key, mitred margin, trimmed joist.

Give weights per cubic foot to the following :—

Fir (Baltic)	lbs.....
Lime plaster	lbs.....
Oak (Austrian)	lbs.....
Teak (Moulmein)	lbs.....
Steel	lbs.....

C.S.I. 1940.

163. Draw to a scale of 1 inch to 1 foot half plans at bottom and top and a half elevation of a steel roof truss for a roof of 45 degrees pitch and a clear span of 12 feet between 13½-inch brick walls. Show the roof covering to consist of 7″ × 3″ purlins, 4″ × 2″ rafters, 1¼ inch rough boarding and felt. The slates and battens and the finishings at eaves and ridge are not required to be shown.

The members of the truss are to be as follows :—

Principal rafters and tie ...	3″ × 2½″ × ½″ angle.
Struts and king piece ...	2″ × 2½″ × ½″ angle.
Purlin cleats	6″ × 3″ × ⅜″ × 6″ angle.
Gusset and bearing plates	⅜ inch thick.
Rivets and bolts	⅝ inch diameter.

Draw to half full size in plan, section and elevation the joint at heel of truss, to show full construction and manner of securing to padstone.

(NOTE.—Centres of rivets or bolts only need be shown.)

Dimension all the members and parts, and indicate in red pencil the diagram or setting out lines for each member on one half of the truss.

Mark in red pencil where the following occur :—

Cleat, countersunk rivet, gusset, pitch, rag bolt.

C.S.I. 1940.

164. Draw to a scale of 1½ inch to 1 foot the half elevation of a king post roof truss for a roof of 30 degrees pitch and a clear span of 8 feet between 13½ inch brick walls. Show

purlins and rafters. Draw all the joints in detail. (NOTE.—Lengths of tie beam, principal rafter, king post and struts may be broken, in order to reduce space; but all the joints must be drawn in proper relationship to each other as they occur on the truss.) Indicate all ironwork, bolts, straps, etc. Give a cross section of each member.

Draw in isometric projection to a scale of 3 inches to 1 foot a bridle joint between strut and principal rafter when separated.

Dimension all the members and parts.

Mark in red pencil where the following occur :—

Bridle joint, notching, oblique tenon, stub tenon, three-way strap. C.S.I. 1940.

165. Explain the use of propeller, pressure, and circulating fans used in ventilating buildings. How do you calculate the discharge from propeller fans ? C.S.I. 1940.

166. Describe the upward and downward methods of ventilation for public buildings, and explain the conditions which cause draughts in these installations. Make a sketch section of a public building with an example of upward ventilation. C.S.I. 1940.

167. State :—

(*a*) What Dr. Angus Smith's solution is used for.
(*b*) What material it is used on.
(*c*) How it is applied.
(*d*) The most important detail in the process of applying it. C.S.I. 1940.

168. On inspecting the intercepting chamber of a large school it is found that the trap is full of solids and paper, which are apparently retained in the trap although the liquid passes through.

The drain is 6 inches; the chamber is brick-built and rendered throughout and measures 3′ 9″ × 3′ 0″ × 4′ 6″ deep; there is a good fall to the drain, but not excessive; the invert of the chamber is 6 inches half-round channel throughout formed with a 24-inch length of straight channel the lower end of which is continued by a short sharp channel bend of about 120 degrees which then discharges direct into the interceptor; there are no branches to the chamber.

Explain why this trap does not get flushed properly; and state how the chamber can be re-designed to remedy the trouble. C.S.I. 1940.

169. If a 4-inch drain, flowing half-full at a velocity of 4½ feet per second, discharges 4,416 gallons per hour, how many gallons per hour will a 6-inch drain discharge when similarly flowing half-full at the same velocity ? C.S.I. 1940.

170. It is required to apply a water test to a new length of 4-inch drain which is 330 feet long.

The ground surface at the upper end is 99·16 above O.D. and the invert of the pipes at that point is 5 ft. 6 in. below the surface ; the ground surface at the lower end is 85·91 above O.D. and the invert of the pipes at that point is 2 ft. 3 in. below the surface.

The contractor has been ordered to plug the lower end of the drain, and to fill it with water as far as possible, the surveyor will then insert a drain stopper which has an air vent tube in the top end and will then complete the filling of the drain through a bucket gauge attached to the central tube of the drain stopper ; the bucket gauge itself will be supported above ground at the upper end of the drain so that the water in the gauge of the bucket stands exactly at 3 ft. above the ground surface.

(The specification provides that the test shall not exceed 10 lb. pressure per sq. in., and also requires the contractor to provide temporary storage of water of 200 gallons for the purpose of carrying out the test.)

The contractor has protested against this proposed test and has formally objected on the grounds that the test is excessive and that there will not be sufficient water on the works to carry it out.

The matter has been referred to you for decision as to whether the contractor's objections are valid or otherwise.

State :—

(*a*) The greatest pressure per square inch that will be exerted on any part of the pipes by this test.

(*b*) The amount of water required to carry out the test, when allowing an excess amount of 10 gallons over the actual contents of the drain for the contents of the bucket gauge and connecting piping and for spillage in filling.

(*c*) Your decision as to whether either of the objections must be upheld or otherwise.

171. (*a*) A gang of men are hauling up a load of one ton by means of a rope passing over a single sheave pulley lashed to a beam. The ganger notices that the lashings require to be reinforced. As it is not desirable

to lower the load he directs the men to secure the lifting rope to the load. What effect will this have upon the stress in the lashing ?

(*b*) A picture weighing 100 lbs. is hung by a single cord passing over a hook. What is the tension in the cord when its length is so adjusted as to be inclined to the horizontal at

(1) 30 degrees ?
(2) 45 degrees ?
(3) 60 degrees ?

(*c*) A rod of uniform section weighing 8 lbs. rests on two supports A and B 8 ft. 0 in. apart. At points 3 ft. 0 in. and 6 ft. 0 in. from A it carries loads of 8 lb. and 4 lb. respectively. What are the reactions at A and B ?

(*d*) If the rod be extended 2 ft. 0 in. over B to C, the cantilever BC carrying an equally distributed load of 4 lbs. and a concentrated load of 6 lbs. being hung on the end C, at what point will the whole balance in a sling ?

(*e*) Define briefly the following terms as applied to reinforced concrete :—

(*a*) Lever arm.
(*b*) Point of contraflexure.
(*c*) Modular ratio.
(*d*) Hooped reinforcement.
(*e*) Effective diameter of columns.

C.S.I. 1940.

172. A stanchion base 2′ 0″ × 2′ 0″ carries a load of 100 tons. Design suitable granite and concrete base to transmit load to earth without exceeding 12 tons and 6 tons per square foot respectively on concrete earth.

C.S.I. 1940.

173. The stanchion in Question 172 consists of two 10 by 5 rolled joists with 12″ × ½″ flange plates. Design base with gusset plates having sufficient rivets to transmit load to base plates without exceeding the following stresses in tons per square inch :—

Single shear	5·5
Double ,,	9·625
Bearing	11.

C.S.I. 1940.

174. A girder AD 24 ft. 0 in. clear span carries an equally distributed load including its own weight of 1 ton per foot run.

At B, 9 ft. o in. from A it carries a concentrated load of 8 tons, and at C, 18 ft. o in. from A, a concentrated load of 4 tons.

(*a*) What are the reactions at A and D ?
(*b*) Draw the shear diagram.
(*c*) Determine the position of maximum bending moment.
(*d*) Ascertain its amount in inch tons by calculation from A and D.
(*e*) Determine the section modulus required consistent with a limiting fibre stress of $7\frac{1}{2}$ tons per square inch.

C.S.I. 1940.

175. A steel roof truss 30 ft. o in. span 10 ft. o in. rise with two purlins on each slope is to be designed to carry vertical loads of 15 cwt. assumed to include wind pressure on each purlin point and ridge.

(*a*) Draw stress diagram.
(*b*) Fix approximately appropriate sections of members.
(*c*) Detail joints to scale not less than $1\frac{1}{2}$ inches to 1 foot with $\frac{3}{8}$-inch gusset plates giving number and diameter of rivets to keep stresses per square inch within the limits of Question 173.

C.S.I. 1940.

176. A rolled joist AB weighing 56 lb. per foot run rests on two bearings 16 ft. o in. apart and extends 4 ft. o in. over B to C where it carries a concentrated load of 6 cwt. The cantilever BC also carries a uniformly distributed load of 4 cwt. Concentrated loads of 8 cwt. and 4 cwt. are carried at points 6 ft. o in. and 12 ft. o in. from A.

(*a*) What are the reactions at A and B ?
(*b*) Draw the shear diagram.
(*c*) What is the nature and amount of the bending moment at B ?
(*d*) Determine the position and amount of the maximum bending moment.
(*e*) Draw the bending moment diagram. C.S.I. 1940.

177. Draw to a scale of $\frac{1}{2}$ inch to 1 foot :—

Vertical section of 11-inch external brick cavity wall, showing the arrangement of the damp-proof course at :—

(*a*) Level of concrete roof with 9-inch parapet wall finished with stone coping.
(*b*) Head of casement window, which is set back $4\frac{1}{2}$ inches from external face of brickwork.
(*c*) Foundation level, the building having wood floor supported on timber joists.

Suitable hatch sectional brickwork, stonework and concrete. Give the names of four different materials for damp-proof courses. C.S.I. 1942.

178. Draw in isometric projection the following joints in carpentry :—

(*a*) Tusk tenon (figure on exact proportional dimensions).
(*b*) Housed joint.
(*c*) Cogged joint.
(*d*) Notched joint.
(*e*) Halved joint.
(*f*) Dovetail halving.
(*g*) Stub tenon. C.S.I. 1942.

Scale 1 inch to 1 foot.

179. Draw a cross-section through the lead gutter formed at the intersection of a slated roof and the top side of a brick chimney stack. Scale $1\frac{1}{2}$ inches to 1 foot.

At the junction of a slated roof with the side of a brick chimney stack, the building is kept watertight by means of lead soakers and a stepped flashing. Show by isometric sketch this construction and give dimensions of lead soaker. Scale $\frac{3}{4}$ inch to 1 foot. C.S.I. 1942.

180. Draw the external and internal elevations and plan of a framed and braced door 2′ 6″ × 6′ 6″ × $2\frac{1}{4}$″. Name and figure sizes of various members. Scale $\frac{1}{2}$ inch to 1 foot. C.S.I. 1942.

181. Draw the following details of a king post roof truss having a span of 20 feet and a pitch of 30 degrees. Give names and sizes of the various members. Scale $\frac{3}{4}$ inch to 1 foot :—

(*a*) Head of the king post.
(*b*) Foot of the king post.
(*c*) Junction of purlin with principal rafter.
(*d*) Junction of principal rafter with tie beam, showing eaves gutter, the back of which projects 15 inches from the external face of the 14-inch wall. C.S.I. 1942.

182. Draw elevation and plan of stud partition between two rooms 12 feet wide by 10 feet high, with two-panelled door, 2′ 8″ × 6′ 8″ in centre. One half of the elevation to show the structural members of the partition, the other to show the finished surface with cornice, picture rail and skirting. Figure on the various sizes. Scale $\frac{1}{2}$ inch to 1 foot. C.S.I. 1942.

183. Briefly describe and illustrate with sketch if necessary what is meant by the following terms :—

(*a*) Entablature.
(*b*) Bressummer.
(*c*) Effective span.
(*d*) King closer.
(*e*) Intrados.
(*f*) Cavetto.
(*g*) Purlin. C.S.I. 1942.

184. State to what pitch a roof should be constructed when covered with each of the following materials :—

(*a*) Slating. (*c*) Lead.
(*b*) Plain tiling. (*d*) Asphalt.

What is the size of the following :—

(*e*) Duchess slate.
(*f*) Countess slate.
(*g*) Plain roofing tile.

What distance apart should the centres of the following be placed :—

(*h*) Rolls of lead flats.
(*j*) Rolls of zinc flats.
(*k*) Drips on lead flats.

What weight of lead should be used for :—

(*l*) Flat roofs.
(*m*) Flashings. C.S.I. 1942.

185. Draw to a scale of 1 inch to 4 feet the plan of the joists of an upper floor room in an ordinary domestic building. The room is 16′ × 12′ and has a fireplace opening 2 ft. 6 in. wide with a 9-inch flue on each side, projecting 13½ inches, midway in one of the longer sides. Show the trimming round the hearth and name and give the scantlings of the timbers. C.S.I. 1942.

186. Give the minimum inclination for roofs with the following coverings :—

(*a*) Lead.
(*b*) Ordinary slating.
(*c*) Pantiles.
(*d*) Plain tiling. C.S.I. 1942.

187. Draw to a scale of 1 inch to 1 foot, the section through the head of a casement window frame, in a 11-inch cavity wall. The brick opening is formed with a half brick straight arch externally and a concrete lintel internally. C.S.I. 1942.

188. Draw to a scale of 3 inches to 1 foot, plan through one side of a 1½-inch deal panelled door, hung to rebated lining, set in a 4½-inch brick partition wall, plastered both sides. Show all details. C.S.I. 1942.

189. Sketch freehand the following types of raftered roofs, indicating the maximum span of each :—

(*a*) Lean-to.
(*b*) Couple.
(*c*) Couple close.
(*d*) Collar. C.S.I. 1942.

190. Give the constituents and proportions of the following :—

(*a*) Concrete in ordinary foundations.
(*b*) Concrete in reinforced lintels.
(*c*) Backing for glazed wall tiling. C.S.I. 1942.

191. Describe the following (with sketches, if desired), and state their respective purposes :—

(*a*) Stone template.
(*b*) Soldered dot.
(*c*) Roll (in lead work).
(*d*) Lead rack.
(*e*) Apron piece. C.S.I. 1942.

192. Draw to scale of 8 feet to 1 inch ground and first floor plans, front elevation, side elevation and section of small, detached village shop. The width between external side walls is to be 20 feet in the clear. The ground floor is to consist of shop, occupying full frontage by 13 feet depth, with staircase behind, 3 feet wide, to run from side to side between partition walls ; behind staircase parlour and kitchen 13 feet in depth. The first floor is to consist of two bedrooms, bathroom and W.C. C.S.I. 1942.

193. Show your calculations to arrive at the weight to be borne by R.S.J. over shop front, supporting front wall and roof over, and half floor of first floor room ; the first floor joists to run from front to back. C.S.I. 1942.

194. Draw section front to back through the ground floor and foundations of a terrace house, two rooms in depth, divided by 4½-inch brick partition ; the front room to have timber joist floor with sleeper walls, etc., and the back room to have solid floor formed with quarry tiles on concrete. Every step should

be taken to prevent damage by exposure, rising dampness or decay. The drawing may be broken so that the three main portions of the floor are reasonably close up. C.S.I. 1942.

195. Draw to a scale of 1 inch to 1 foot :—

(*a*) A section and part elevation of a stone cornice, 1 ft. 9 in. deep, with blocking course 1 ft. 6 in. high. Show water or saddled joint to cornice and dowel to blocking course.

(*b*) The junction at the head of a queen post with principal rafter ; show straining beam, purlin, common rafter, three-way strap, etc.

(*c*) Plan and section of a chimney breast and hearth on the first floor of a dwelling-house : show trimmer and trimming joists, etc., and cornice to ground floor room. C.S.I. 1942.

196. Draw to a scale of $\frac{1}{8}$ inch to 1 foot, a steel roof truss, suitable for a span of 40 feet. Draw to a large scale detail of joints at ridge, and at foot of principal rafter showing gutter wall, etc. C.S.I. 1942.

197. Draw to a scale of $\frac{3}{4}$ inch to 1 foot, a plan, elevation and section of an inward opening french casement with fan-light over, in a stone wall, 2 feet thick. Opening to be 3 ft. 9 in. wide by 9 feet high.

Casement to be hung folding ; finish inside with moulded and panelled linings and architrave. C.S.I. 1942.

198. Draw to a scale of $\frac{1}{2}$ inch to 1 foot an elevation showing the construction of a timber-framed partition across a room, 18 feet wide, on the second floor of a dwelling-house, with a doorway opening, 3′ 6″ × 7′, in the centre. Height, floor to floor, 11 feet. The partition to be constructed to support the floor above and below. Figure the sizes. C.S.I. 1942.

199. Draw to a scale of $\frac{1}{4}$ inch to 1 foot, a ground floor and first floor plan of a small house.

The plan to be similar to the line drawing below.

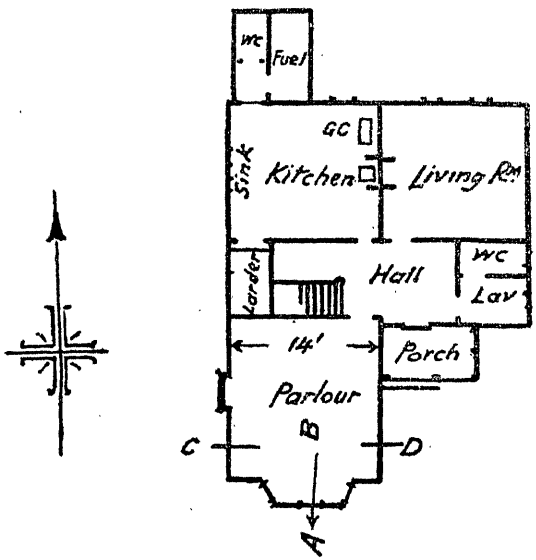
The first floor to have three bedrooms, bathroom, W.C., and linen cupboard.

Walls on ground floor to be 11½ inches hollow, and on first floor 9 inches, rough-casted.

The parlour to be 14 feet wide ; the bay window to have a lead flat roof : the main roof to be tiled and the gable to have a barge board. C.S.I. 1942.

200. Draw detail of the front elevation to a scale of $\frac{1}{4}$ inch to 1 foot. C.S.I. 1942.

201. Draw sections A—B and C—D where indicated on plan to a scale of $\frac{1}{4}$ inch to 1 foot. C.S.I. 1942.



GROUND · FLOOR · PLAN

202. Draw to a scale of 1 inch to 1 foot a plan, section and external elevation of a window opening in a 11-inch brick cavity wall, sizes between brickwork to be 3′ × 5′ 6″, with flat gauged face arch and York stone cill.

Opening to be filled with a pair of 2-inch wrought deal double-hung sashes divided into squares for glazing, and complete with deal-cased frame, sash weights, oak cill and all finishings.

Show how you would prevent penetration of dampness above and around the opening. C.S.I. 1942.

203. Draw to a scale of 1 inch to 1 foot plans showing alternate courses, section and elevation of a four-flue brick

chimney stack in Flemish bond passing through one slope of a slated roof and extending 8 feet above, finished with a necking and three oversailing courses, and 24-inch terra cotta pots.

The roof slope is 45 degrees, rafter 5″ × 2″, covered with 1-inch rough boarding.

Show the trimming of roof, and method you would adopt to avoid penetration of damp at the junction with the roof slope, and down the chimney stack. C.S.I. 1942.

204. Only two of the following questions to be attempted :

(*a*) Draw to a scale of $\frac{1}{4}$ inch to 1 foot the side elevation of a raking shore for a wall of a three-storey building 30 ft. high, 14-inch work to first two floors and 9-inch above.

Show the angle of shore and name and dimension the various parts.

(*b*) Draw to a scale of $\frac{1}{2}$ inch to 1 foot elevation of a king post roof truss for a 20 feet span, the roof to be covered with feather-edged $\frac{3}{4}$-inch rough boarding and tiled.

(*c*) Draw to a scale of $\frac{1}{2}$ inch to 1 foot section through the ground floor of a building 20 feet wide having an 18-inch external brick wall with footings and concrete base, the inside wood floor is of joist and boarded construction on sleeper walls and 18 inches above ground level. C.S.I. 1942.

205. Draw to a scale of 1 inch to 1 foot the plan, a true cross section through wall and roof, and a section on the plane of the hip rafter, at the corner of a hipped roof over a building having 13½-inch brick walls.

The roof to be constructed with 4½″ × 3″ plates, 5″ × 2″ rafters, 7″ × 3″ purlin set, 6′ 6″ up slope from plate, 11″ × 3″ hip rafter and 5″ × 4″ framed dragon tie.

The pitch or slope of the roof is 45 degrees, the span between walls is 16 feet and rafters project 6 inches at eaves with shaped ends.

Show the usual method of supporting the hip at the corner of the walls including all the various carpenter's joints.

Indicate by dotted lines on plan the position for hip rafter and the jack rafter set at 16-inch centres.

Dimension all the members and parts.

Give a line diagram to show method of ascertaining the pitch or slope of the hip rafter.

Mark in coloured pencil where the following occur :—

Dovetailed halved joint, halved joint, jack rafter, oblique tenon, tusk tenon.

Give weights per cubic foot to the following :—

Common brickwork in cement mortar ... lbs.....
Pitch pine ... ... lbs.....
Red deal ... ... lbs.....
Slate ... ... lbs.....
Wrought iron ... ... lbs.....

C.S.I. 1942.

206. Draw to a scale of 1 inch to 1 foot the half plan at one end and a cross-section of the floor of an upper room measuring 18′ 9″ × 12′ 9″ inside of 13½-inch brick walls which are increased to 18 inches thick to the room below.

The floor is constructed with 4½″ × 3″ plates, 9″ × 2″ joists, 9″ × 3″ trimmer and trimming joists, and 1-inch solid strutting.

There is a well hole 4 feet square, between trimming joists, in the centre of the half floor.

Indicate on one half of plan between walls the carpentry construction and on the other half of plan the manner of finishing the 1¼-inch flooring about the well hole which has 9½″ × 1″ tongued linings, 3″ × 1″ architrave on plastered ceiling and 4″ × 1¼″ mitred margins to flooring.

Dimension all members and parts.

Draw to half full size scale an isometric projection of the joint used to secure the ends of the trimmer joists to trimming joists, name and give proportions of all parts of this joint.

Mark in coloured pencil where the following occur :—

Dovetailed halving, key, mitre, trimmer joist, tusk tenon joint. C.S.I. 1942.

207. Draw to a scale of one-eighth full size the plan, section and elevation of a 2½-inch thick framed, ledged and braced door size 2′ 9″ × 6′ 9″ inside of rebates of a 5″ × 3″ rebated and moulded frame with ½-inch backings set in a 4½-inch outside reveal having 2¼-inch recess in a 13½-inch wall finished with 1-inch linings and architraves to plastered walls inside.

The door to have 5-inch styles and top rail, 8-inch middle and bottom rails and 5-inch braces, and to be sheeted on outside from top rail to bottom of door with tongued and grooved and V-jointed battens not exceeding 3 inches wide and 1 inch thick.

Indicate method of securing frame at bottom and sides of posts and at head into floor and brickwork respectively.

Indicate on one style the joints used to frame the ends of the three rails into the styles, noting that the middle or lock rail is to take a 6-inch mortice lock.

Show bonding for brickwork, alternate courses, on each side for English Bond and a flat arch with lintel at back at head.

Dimension all the members and parts.

Mark in coloured pencil where the following occur :—

Barefaced tenon, dowel, haunched tenon, horn, scribing.

C.S.I. 1942.

208. Draw to a scale of 1 inch to 1 foot the plan, cross and longitudinal sections of a manhole on a 6-inch diam. main drain which receives one 4-inch diam. branch drain on each side.

The manhole is to be 2′ 3″ × 3′ × 4′ deep to invert.

Indicate foundations, benchings, glazed invert channels and iron cover (24″ × 24″ in clear) and means of access in shaft.

Show bonding of brickwork for alternate courses on one half of plan in English bond for 9-inch thick walls.

Dimension all members and parts.

Indicate direction of flows.

Mark in coloured pencil where the following occur :—

Benching, grease and sand joint, oversailing courses, relieving arch, step irons. C.S.I. 1942.

209. (*a*) A triangular sign board weighing ½ cwt., with one side vertical and top edge horizontal of uniform thickness and weight is hung on two chains from hooks at points 8 inches from each end of top edge, 4 feet long. What proportion of the weight is taken by each chain ?

(*b*) State the mathematical relations between :—

(1) Bending moment (BM).
(2) Moment of resistance (MR).
(3) Extreme fibre stress (f).
(4) Section modulus (Z).

(*c*) A load of 16 cwt. is hung on a rope 20 feet long. What force is required to pull it 12 feet in a horizontal direction, and what would be the tension in the rope in that position ?

(*d*) Define the terms :—

(1) Elastic limit.
(2) Modulus of elasticity.
(3) Modular ratio.

Give examples of (2) and (3).

(*e*) State any rules with which you are acquainted as to pitch of rivets and distance from edge of plate.

210. A square reinforced concrete column 13″ × 13″ overall and 15 feet effective length carries a load of 48 tons. On account of slenderness permissible loading is 80 per cent. of nominal loading, *i.e.*, it must be designed for a load of 60 tons. Assume adequate lateral hooping and design vertical reinforcement for permissible stresses on steel and concrete of 16,000 and 600 lb. per square inch respectively with $m = 15$.

C.S.I. 1942.

211. A reinforced concrete balcony consisting of a slab floor 6 inches thick reinforced in tension with ½-inch diameter rods (0·1963 square inch) at 5-inch centres with ½-inch cover is carried on cantilever rib brackets at 6-foot centres.

The weight and superload of slab floor is 150 lbs. per square foot. Assume slab floor to be continuous over ribs and therefore subject to a bending moment under distributed load of WL/12 when L = 6 feet. Assume adequate shear reinforcement, that $m = 15$ and that $n_1 = \sqrt{m^2r^2 + 2mr} - mr$.

Calculate maximum stresses on steel and concrete in slab.

C.S.I. 1942.

212. Calculate the moment of inertia and section modulus of two 10″ × 1″ plates, 12 inches apart in the clear in a compound girder deducting two ⅞-inch holes for ¾-inch rivets in each plate. C.S.I. 1942.

213. A steel roof truss built up from 3″ × ⅜″ flats and 3″ × 2½″ × ⅜″ angles is 40 feet span and 30 degrees pitch. It carries a ridge and four purlins equally spaced, with vertical members under each purlin and ridge. A horizontal main tie also carries the bearers of a loft floor parallel with the purlins at 6 ft. 8 in. centres. The ridge and purlin points each carry a roof load of 1 ton and the bearers a floor load of 3½ tons each. Both loadings include weight of truss, bearers, roof and floor. Complete bracing, draw stress diagrams and indicate nature and amount of load on each member.

Sketch to scale of not less than 1½ inches to 1 foot details of joints. Permissible stresses in tons per square inch: tension 7·5; bearing 11·0; single shear 5·5; double shear 9·625.

C.S.I. 1942.

214. A riveted plate girder, consisting of web, flange plates and flange angles and 4 feet deep over flange angles carries loads causing a reaction of 48 tons at one bearing. The web plate at the bearing is ⅜ inch thick and the flange angles are 3″ × 3″ × ½″. Assuming horizontal and vertical shear to be the same

in any bay between stiffeners at what pitch should $\frac{3}{4}$-inch rivets be spaced in the bay next to the bearing ? Permissible stresses as in Question 213. C.S.I. 1942.

215. A girder AC 28 feet long, is carried on two bearings at A and at B, 20 feet from A. It carries a uniformly distributed load, including its own weight, of 5 cwt. per foot run. It also carries concentrated loads of 2 tons each at four points respectively 5 feet, 10 feet, 15 feet and 20 feet from A. At the end of the cantilever BC it carries a concentrated load of 7 tons 4 cwt. (7.2 tons). Calculate the centre of gravity of loads and from it the reactions at A and B. C.S.I. 1942.

216. Draw the shear diagram of the loaded girder in Question 215. Calculate the hogging bending moment at B, the maximum sagging moment on the span AB and state the distance from A at which it occurs. Calculate the section modulus required to restrict maximum fibre stress to $7\frac{1}{2}$ tons per square inch.

217. Draw to a scale of 1 inch to 1 foot cross-section at the eaves of a framed timber roof with stone parapet showing full details of construction of the mason, joiner, slater, and plumber works. C.S.I. 1942.

218. Draw to a scale of $\frac{3}{4}$ inch to 1 foot plan and elevation of a 2-inch thick french casement window, to open outwards, also to a scale of 3 inches to 1 foot, fully detailed sections through the jamb, sill and lintel. C.S.I. 1942.

219. Draw to a scale of 2 inches to 1 foot cross-section through a "Buchan" trap with manhole before the house drains enter the main sewer. Show full details of construction and method of ventilation. C.S.I. 1942.

220. The slates on a roof are 24″ × 12″. An average slate weighs 7 lb. The slates are laid with a 4-inch lap. State :—

(1) The number of slates ; and
(2) The weight of slates required to cover a roof with an area of 100 square feet.

Draw to a scale of 2 inches to 1 foot cross-section through four rows of slates from eaves course. C.S.I 1942.

221. Draw to a scale of $\frac{1}{4}$ inch to 1 foot plan of a timber "Geometrical" stair, also to a scale of 2 inches to 1 foot cross-section through two of the steps showing full details of construction. C.S.I. 1942.

222. Draw to a scale of 1½ inches to 1 foot cross-section through a timber cistern lined with lead. Show the necessary piping and fittings required to give the supply of hot and cold water throughout the house. C.S.I. 1942.

223. An 18-inch thick brick wall with the usual footings and concrete is to be under-pinned to a depth of 6 feet. Describe fully how this should be done, and draw a sketch elevation and section of the first portion of the completed work, showing the method of construction.

Old footings to be shown by dotted lines. C.S.I. 1942.

224. Sketch flying shores necessary in pulling down one building in a continuous row.

The building has a 20 feet frontage and is 30 feet deep. The buildings on either side which are to remain standing are 30 feet high.

Show details at joints and mark on sizes of timbers required. C.S.I. 1942.

225. Draw to a scale of 3 inches to 1 foot cross-section through a pedestal wash-down W.C. with trap, lead soil pipe, connected to cast iron soil pipe, also detail half full size at junction of 4-inch cast iron soil pipe and 6-inch diameter fireclay drain. C.S.I. 1942.

226. Describe fully the method of supplying hot and cold water to a house containing three rooms, kitchen, scullery and bathroom.

Illustrate your answer by sketches showing piping, etc., beginning from water main in street. C.S.I. 1942.

227. Explain the meaning of the following terms and illustrate your answers by sketches :—

(1) Joggle joint ; (4) Dragon tie beam ;
(2) Parpend ashlar ; (5) Flitch beam.
(3) Boot boiler ; C.S.I. 1942.

228. Draw to a scale of 1 inch to 1 foot plans of two consecutive courses of a brick pier, 3½ bricks square, built in Flemish bond. C.S.I. 1943.

229. Draw to a scale of 2 inches to 1 foot section through a timber opening rooflight in a flat roof covered with Limmer asphalt.

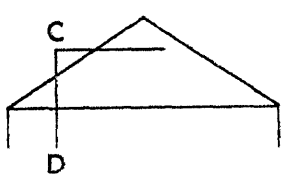
Show full details of construction relative to the joiner, glazier, plumber and asphalt works. C.S.I. 1942.

230. Draw half full size sections through the jamb sill, countercheck and lintel of a 2-inch thick double hung sash and case window, showing full details of construction.
C.S.I. 1942.

231. Draw to a scale of $\frac{1}{2}$ inch to 1 foot elevation and section of a timber centre, for a semi-circular opening in an 18-inch thick brick wall.
The opening to be 8 feet wide and 10 feet high from ground level to spring of arch.
Mark on sizes of timber you would adopt and show method of construction from ground level. C.S.I. 1942.

232. The outline diagram shows the plans of a dormer window to be inserted in a slated roof. Draw to a scale of $\frac{1}{4}$ inch to 1 foot plan and section, also quarter full size details at A B on plan, also section through sill, lintel and roof at C D.
Show full details of construction of the joiner, slater, plaster and plumber works.
The window to be 2 inches thick double hung sash and case. C.S.I. 1942.





233. Draw to a scale of $\frac{1}{4}$ inch to 1 foot plan of timber dog-legged stair; also to a scale of 3 inches to 1 foot cross-section through two of the steps showing full details of construction.
Explain the meaning of the following terms and illustrate your answers by sketches :—

(1) Parpend ashlar; (2) Stirrup strap;
(3) Heel strap; (4) Flitch beam; (5) Toby;
(6) Standing waste and washer in cistern.
C.S.I. 1942.

234. Draw to a scale of $\frac{3}{4}$ inch to 1 foot plans and elevation of a french casement window to open outwards, also to a scale of 3 inches to 1 foot full detail sections through jamb, sill and lintel. C.S.I. 1942.

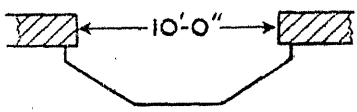
235. Explain what is meant by " dry rot " and state your views as to what should be done to eradicate the rot which has appeared at the floor and skirting at the partition wall.
C.S.I. 1942.

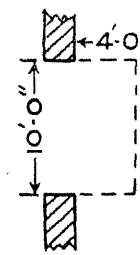
236. Draw to scale of 1 inch to 1 foot cross-section through back-to-back fireplaces in a 2 ft. 6 in. thick brick wall, on an upper floor.
Show full details of construction of the fireplaces, flues, hearth, flooring, joisting, deafening, and lath and plaster ceiling. C.S.I. 1942.

237. A billiard room, 24′ × 18′, is to be covered by a flat roof with cupola light, 12′ × 6′, over the billiard table.
The roof and upstand at cupola are to be of reinforced concrete.
Draw to a scale of $\frac{1}{2}$ inch to 1 foot cross-section through roof and cupola, showing full details of construction at parapet, roof, roof covering and cupola. C.S.I. 1942.

238. A cistern is required to hold 1,000 gallons of water. What should be the length, breadth and depth, and of what material should it be constructed if used for storing drinking water.
What would be the weight of water in the tank when full, and what pressure per square foot on the bottom of the tank ?
Show by sketches the fittings required in the cistern for the supply and distribution of the water. C.S.I. 1942.

239. Draw to a scale of $\frac{3}{4}$ inch to 1 foot plan and section of a timber oriel window with 2-inch thick double-hung sash and case window. The opening in the first floor as per sketch, 10 feet wide between wall surfaces. The projection is 4 feet from face of wall.
Show full details of construction. C.S.I. 1942.





240. Give the weights of :—

(1) Brickwork in cement per cubic foot.
(2) Portland stone per cubic foot.
(3) Aberdeen granite per cubic foot.
(4) Pitch pine per cubic foot.
(5) Greenheart per cubic foot.
(6) Water per cubic foot. C.S.I. 1942.

241. The slates on a roof are 24" × 12". An average slate weighs 7 lb. The slates are laid with a 4-inch lap. State : (1) the number ; and (2) the weight of slates required to cover a roof with an area of 100 square feet. Draw to a scale of 3 inches to 1 foot cross-section through four courses of slates beginning at the eaves. C.S.I. 1942.

242. Draw to a scale of $\frac{1}{4}$ inch to 1 foot plan and section of a timber [illegible] also to a scale of 3 inches to 1 foot cross-section [illegible] of the steps showing full details of construction. C.S.I. 1942.

243. Draw to a scale of $\frac{3}{8}$ inch to 1 foot elevation and section of a steel trussed wood beam suitable for a timber foot bridge of 30 feet span.

Show full details of construction, also mark on sizes of timber beam and members of steel truss. C.S.I. 1942.

244. Draw full size section through a waste pipe from a teakwood sink, showing full details, with overflow connections to waste pipe and trap complete. C.S.I. 1942.

245. Explain the meaning of the following terms and illustrate your answers by sketches :—

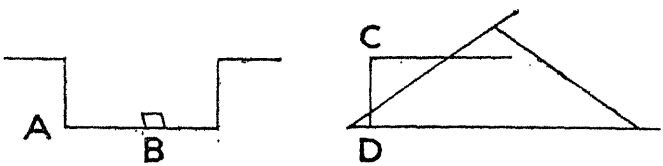
(1) Belfast roof ; (2) Brass union ; (3) Parpend ashlar ;
(4) Saddlejoint on stone cornice ; (5) Iron dog ;
(6) Lead-welted joint. C.S.I. 1942.

246. Draw to a scale of $\frac{1}{2}$ inch to 1 foot half cross-section of a timber hammer beam bound roof truss suitable for a church, span 30 feet. Show full details of construction. C.S.I. 1942.

247. Draw half full size section through a 2-inch thick double-leaved swing door in a 9-inch brick partition.

Section to be through jamb and centre stiles showing full details of construction. C.S.I. 1942.

248. The outline diagram shows the plan of a dormer window to be inserted in a slated roof.



Draw to a scale of $\frac{1}{2}$ inch to 1 foot plan and section also quarter full size details at A and B and C and D.

Show full details of construction of joiner, slater, plaster and plumber works.

The windows to be of the french casement type to open outwards. C.S.I. 1942.

249. Draw to a scale of $\frac{1}{4}$ inch to 1 foot a plan and elevation of a stair with a continuous string and rail, a well 1 foot diameter, and a cut string. To a scale of 3 inches to 1 foot draw also a section through three of the steps showing full details of the construction.

250. A masonry niche 4 feet wide in an ashlar wall is semi-circular in plan, and has a semi-circular head. Make inch scale drawings of plans, sections, and elevations showing two methods of constructing the upper part of the niche. R.I.B.A. 1938.

251. A lead-covered flat roof of timber and steel joist construction is to be built over a square hall, 30′ × 30′. The R.S.J.'s are spaced at 10-feet centres. The main structural walls are 14-inch brick with 9-inch parapet walls. In the centre of the flat a rectangular lantern light is to be formed with opening vertical lights.

Draw the cross section through the roof and lantern light to $\frac{1}{2}$-inch scale, and add full-size details of the lantern light. R.I.B.A. 1938.

252. Draw to $\frac{1}{2}$-inch scale, plan and section of a sloping gallery with seats in tiers, and with a straight front constructed of timber and woodwork at the end of a concert hall.

The width of the building is 40 feet, and the depth of the gallery from front to back is 18 feet. Oak posts may be used as supports near the front of the gallery, the under side of which is 10 feet above the floor of the hall. R.I.B.A. 1938.

253. A two-storey domestic building is to be built in a some-

what exposed situation in brickwork, with 11-inch hollow walls and with wood casements 4 ft. 6 in. high in straight reveals. The height floor to floor is to be 9 ft. 3 in. The floors to be boarded on wood joists, the ground floor joists to be $4\frac{1}{2}'' \times 2''$ carried on sleeper walls and concrete bed; the first floor joists to be $9'' \times 2''$.

Draw to 1-inch scale plan at window jamb and section

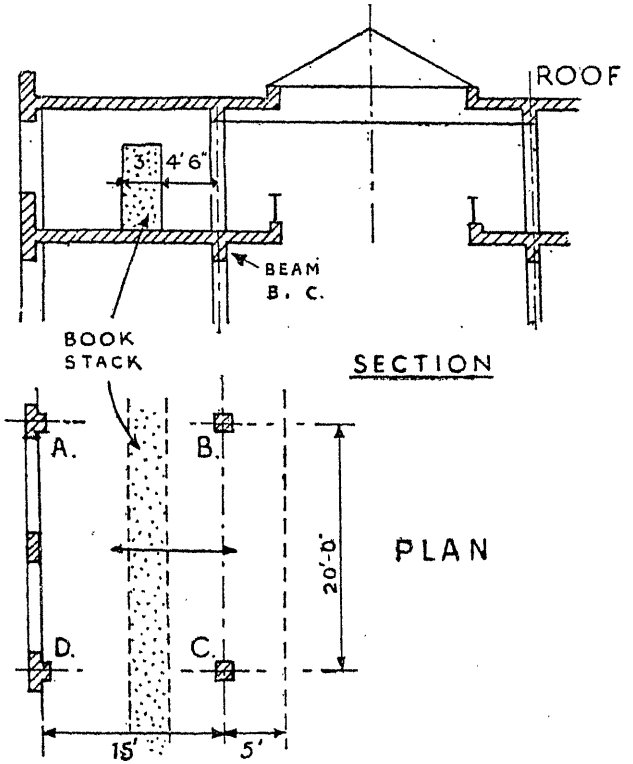


Fig. 2.

through wall showing floors and ground floor window, brick footings, concrete foundations, etc. Add the necessary large scale sketch details. R.I.B.A. 1938.

254. A house near the centre of a terrace of similar houses

is to be taken down and rebuilt, and it is necessary to support the party walls of the adjoining houses.

The houses are of three storeys in addition to a basement. The ground floor level is 2 feet above the pavement and starting with the basement the storeys are 8 feet, 10 feet, 9 feet, and 8 feet high from floor to ceiling. The thickness of the several floors and the flat over the top storey is 12 inches. The party walls are 18 inches thick up to ground floor and 13½ inches thick for the rest of their height up to flat level with 9 inches parapet wall 2 ft. 6 in. above flat level. The depth of the houses from front to back is 35 feet, and the width of each house is 30 feet.

Make drawings to ½-inch scale showing the necessary temporary shoring, etc., with large scale explanatory sketches. R.I.B.A. 1938.

255. Fig. 2 shows in plan and section a portion of the top floor of a steel framed building with stanchions at A, B, C and D. The reinforced concrete slab spans 15 feet and is cantilevered beyond the beam B C and has a raised kerb to which is secured a metal balustrade.

A book stack, 3 feet wide, is to be placed in the position shown, and over the area covered by this stack the *inclusive* floor load is 250 lbs. per square foot. On the remainder of the slab the *inclusive* floor load is 150 lbs. per square foot.

(*a*) Calculate for and select a suitable section for the steel beam B C.

(*b*) Design the floor slab and cantilever.

(*c*) Draw a cross section of this slab to ½-inch scale and show the disposition of reinforcement. R.I.B.A. 1938.

256. One wing of an office building, consisting of ground, first, second and third floors and a flat roof has a double line of steel stanchions and beams defining the central corridor as shown in Fig. 3. Each pair of stanchions has a combined base.

The patent hollow tile floors span from the beams to the external structural brick walls and the *inclusive* load for each of the three floors is 140 lbs. per square foot. The *inclusive* load for the roof slab is 120 lbs. per square foot. Each stanchion from base to first floor consists of an 8″ × 6″ section with one 10″ × ½″ plate to each flange.

Design and draw to 1-inch scale a suitable combined foundation for one pair of stanchions, assuming a safe soil load of 1 ton per square foot. R.I.B.A. 1938.

257. Fig. 4 indicates the plan, at second floor level, of a reinforced concrete staircase, with open well and metal balustrade, in a reinforced concrete framed building. The height

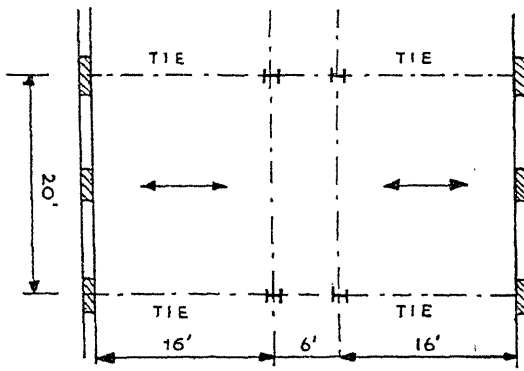


Fig. 3.

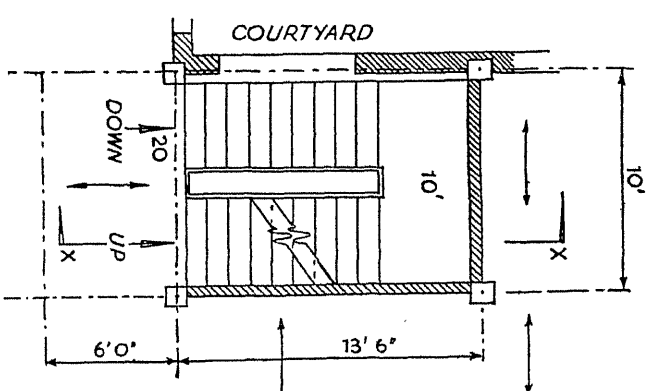
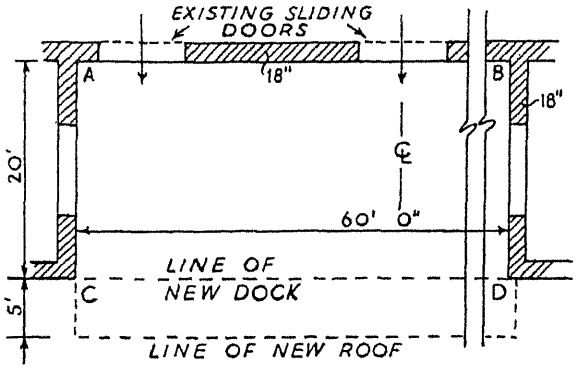


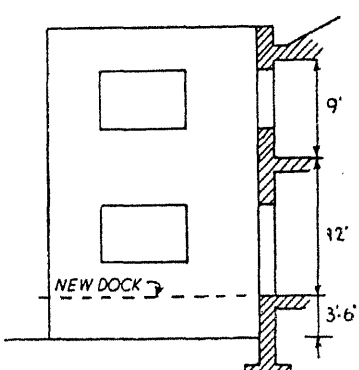
Fig. 4.

from first floor to second floor is 11 feet and the treads and risers are to be finished with compressed cork.

Draw the plan and section x x to ½-inch scale showing the



PLAN



CROSS SECTION

Fig. 5.

method you suggest for supporting the staircase and the position of all reinforcement. No calculations are required.

R.I.B.A. 1938.

258. The rear wall of an existing 2-storey warehouse is set back to form a loading bay 60′ × 20′ as shown in Fig. 5, and

goods are at present despatched through the three 8′ × 8′ sliding doors on line A B.

It is now proposed to extend the dock to the line C D and over it to construct a roof of steel and glass, at about first floor level. The roof is to project over the roadway 5 feet. No isolated supports are permitted, but new piers or stanchions may be placed against the existing walls.

Draw the plan to $\frac{1}{8}$-inch scale and a cross section to $\frac{1}{2}$-inch scale, with full notes, to indicate your suggestions for the new roof and new dock. No calculations are required.

R.I.B.A. 1938.

259. Assuming that you are the Architect for a large reinforced concrete building in which the quality of materials, method of mixing and standard of workmanship are of vital importance, describe the precautions, investigations and tests which you would suggest when preparing your Specification and also during the progress of the work. R.I.B.A. 1938.

260. A ground storey stanchion properly restrained at both ends in position and direction carries a concentric load of 54 tons. The height from ground to first floors is 18 ft. 8 in. It is proposed to use a 9″ × 7″ rolled steel stanchion. Is this a suitable section? Give calculations to support your answer.

A stanchion of the same storey height elsewhere in the building carries a concentric load of 40 tons from upper floors and a first floor beam with a 9-tons end reaction supported by an angle bracket centrally on one flange. Assuming that the stanchion is tied in the other directions at first floor level calculate for and select a suitable section. R.I.B.A. 1938.

261. A set of regulations for the control of Reinforced Concrete Construction is required by the authorities of a large provincial town. State what requirements you would advise them to make under the following headings :—

(*a*) Quality of concrete.
(*b*) Limiting stresses in steel and concrete.
(*c*) Beam design.
(*d*) Column design.
(*e*) Cover of steel. R.I.B.A. 1938.

262. A house costing about £5,000 is to be built on a site where there is no public sewage disposal scheme. A small stream forms one boundary of the site about 200 yards away from the house.

Illustrate by means of a small scale block plan and details to a suitable scale the system you would adopt. R.I.B.A. 1938.

263. Describe briefly the method you would adopt of heating the various rooms in the house referred to in question No. 262.

Assuming that central heating is to be employed for all or some of the rooms, illustrate this system diagrammatically, and give details of the heating units installed in the rooms. If you think it desirable to install central heating, give reasons and describe the alternative methods you would use. R.I.B.A. 1938.

264. Illustrate diagrammatically the hot and cold water supply to the house referred to in question No. 262, assuming that a public water supply is available. Lavatory basins are fitted in each of the bedrooms, and the manner in which you would run hot and cold services to each basin should be indicated. R.I.B.A. 1938.

265. The lavatory block to a factory is a detached single-storey building. Illustrate with sketches and diagrams the fittings you would use and the method of finish adopted for floors, walls, ceilings and internal construction generally. R.I.B.A. 1938.

266. The Assembly Hall in a technical college provides seating accommodation for about 600. The Hall is required for a variety of purposes, including examinations, prize-givings, exhibitions, cinematograph displays, and concerts.

What form of ventilation would you recommend for such a Hall, assuming that no form of mechanical ventilation is required for the remainder of the college building ? You may assume any reasonable system of heating for the remainder of the building. R.I.B.A. 1938.

267. Give in full the following clauses in a specification for a new house :—

(*a*) Concrete foundations.
(*b*) Concrete pavings with granolithic finish.
(*c*) Reinforced concrete lintols. R.I.B.A. 1938.

268. What precautions would you take in a specification to prevent dry rot in :—

(*a*) Wood floors in ground and upper storeys.
(*b*) Ends of roof timbers. R.I.B.A. 1938.

269. Give a general clause describing the lead you would use for the following :—

(*a*) Flat over bay window.
(*b*) Soakers.
(*c*) Stepped cover flashings.
(*d*) Gutters to chimneys. R.I.B.A. 1938.

270. Name four hardwoods in common use in joinery work, and describe the kind of surface treatment you would adopt to develop and preserve the appearance of the wood in each case. R.I.B.A. 1938.

271. Give a list of the several kinds of damp-proof courses in general use. Distinguish those which can be used vertically as well as horizontally, and also those which are suitable for forming a damp-proof course in an old wall. R.I.B.A. 1938.

272. A simply supported timber beam 12″ × 4″ spans 18 feet and carries a uniformly distributed load of 3 cwts. per

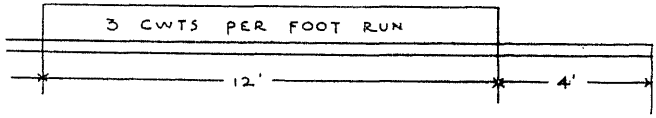


Fig. 6.

foot run over 12 feet of its length as shown on Fig. 6. Allowing a maximum extreme stress of 1,200 lbs. per square inch :—

(*a*) Show by calculations if the beam can safely carry the load.

(*b*) Calculate the maximum stress in this beam at a point 4 feet from the right hand support. R.I.B.A. 1938.

273. A retaining wall cross section is shown in Fig. 7.

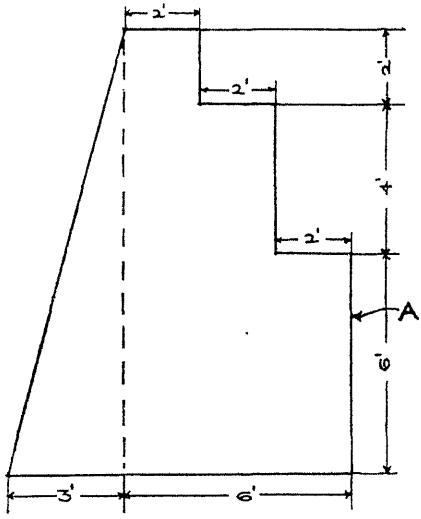


Fig. 7.

Draw this to the scale of $\frac{1}{2}$ inch equals 1 foot. Find by calculation or graphically the position of the centre of gravity of this section and figure the distances of this point from the base and from the vertical face A. Show all graphic construction or full calculations. R.I.B.A. 1938.

274. The frame diagram of a roof truss is shown in Fig. 8 with the loads given in cwts. The truss is supported at A and B. Draw this to the scale of $\frac{1}{4}$ inch equals 1 foot. Draw the stress diagram to the scale of $\frac{1}{4}$ inch equals 1 cwt. Figure the stresses found close to the particular member on the frame

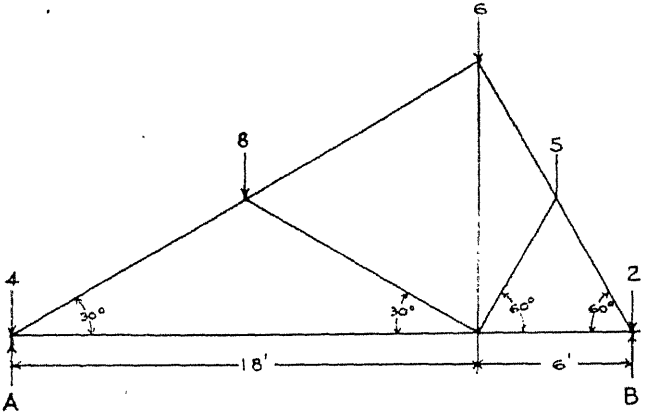


Fig. 8.

diagram and indicate compression by a plus sign and tension by a minus sign. R.I.B.A. 1938.

275. A tension member consisting of double flats (two $\frac{1}{2}'' \times 1\frac{3}{4}''$) is attached to a $\frac{3}{4}$-inch thick gusset plate as shown in Fig. 9. Assuming that the gusset plate is amply strong in tension and so may be neglected for this stress, show by full calculations the maximum safe pull that may be applied to the tension member. All rivets are $\frac{3}{4}$ inch diameter.

Maximum Tension Stress	=	8	tons	per	square	inch.
Single Shear	=	6	,,	,,	,,	,,
Double Shear	=	12	,,	,,	,,	,,
Bearing Stress	=	12	,,	,,	,,	,,

R.I.B.A. 1938.

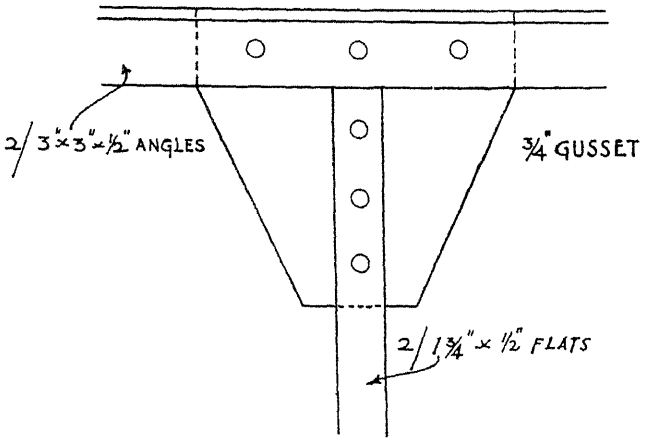


Fig. 9.

276. Explain and define briefly but clearly each of the following terms using sketches or examples where you consider they are necessary :—

(*a*) Radius of gyration.
(*b*) Principle of moments.
(*c*) Moment of inertia.
(*d*) Section modulus.

A beam is in equilibrium under the action of the forces shown

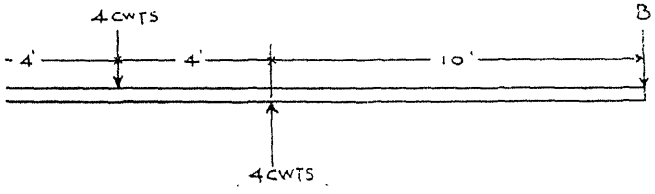


Fig. 10.

in Fig. 10—the arrows show the direction in which they act. Calculate forces A and B and draw the shear force and bending moment diagrams for this beam. R.I.B.A. 1938.

277. Express in terms of W (the load) and L (the span) the maximum bending moments in a simply supported beam loaded with :—

(*a*) A uniformly distributed load over the whole span.
(*b*) Equal point loads situated at third points of the span.
(*c*) Two point loads situated at third points, but one load being three times greater than the other.

Do the calculations algebraically in full, and sketch free hand the bending moment and shear force diagrams in each case.

R.I.B.A. 1938.

278. A scissors type roof truss spanning 32 feet is shown in Fig. 11. The loads are shown in cwts.

(*a*) Draw the frame diagram to $\frac{1}{4}$-inch scale.

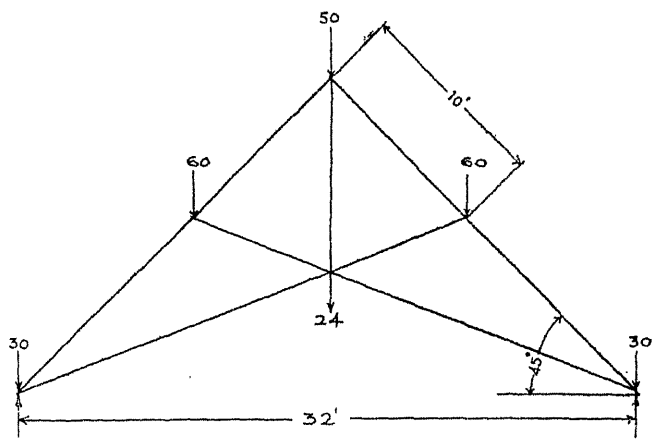


Fig. 11.

(*b*) Draw the stress diagram to scale of $\frac{1}{4}$ inch equals 10 cwts.
(*c*) Figure on the frame diagram close to each member the stress it receives in cwts. and indicate compression by the plus sign and tension by the minus sign.

R.I.B.A. 1939.

279. Define clearly and briefly the following terms, and make sketches where considered necessary :—

(*a*) Bending moment.
(*b*) Section modulus.
(*c*) Moment of inertia.
(*d*) Neutral axis.

R.I.B.A. 1939.

280. The deflection of a simply supported beam carrying a uniformly distributed load is given by the formula deflection $= \frac{5}{384} \frac{WL^3}{EI}$. Calculate the maximum uniformly distributed load that may be carried by a 9″ × 3″ wooden joist spanning 15 feet simply supported, if the deflection is limited to $\frac{1}{325}$ part of the span.

Note W = total load in lbs.
L = span in inches.
E = 1,200,000 lbs. per square inch.
I = Moment of inertia. R.I.B.A. 1939.

281. A house near the centre of a terrace of similar houses has been taken down, and it is necessary to support the party walls of the adjoining houses. The houses are of three storeys in addition to a basement. The ground floor level is 2 feet above the pavement and starting with the basement the storeys are 8 feet, 10 feet, 9 feet, and 8 feet high from floor to ceiling. The depth of the houses from front to back is 35 feet and the width of each house is 30 feet.

Make drawings to ½-inch scale showing the necessary temporary shoring, etc., and add explanatory notes.

R.I.B.A. 1939.

282. A 10″ × 10″ compound stanchion transmits a load of 144 tons to a two-tier grillage by means of a steel slab (bloom) base. The concrete foundation measures 7 feet in one direction.

Calculate the thickness of the bloom base and then design the two-tier grillage and show sketches, given the following permissible stresses :—

Soil	2 tons per square foot
Bloom base	9 tons per square inch
Grillage joists (encased)	12 tons per square inch

Note.—Allow 10 tons for the weight of the completed foundation. R.I.B.A. 1939.

283. Twin rolled steel joists properly bolted together and simply supported at the ends over a span of 24 feet are required to carry a uniformly distributed load of 1 ton per foot over the entire span and concentrated loads of 12 tons and 16 tons at points 6 feet and 18 feet respectively from the same support.

(*a*) Draw the vertical shear force diagram.

(*b*) Calculate the maximum bending moment and state where this occurs; select suitable beams allowing 8 tons per square inch as the maximum permissible stress.

(*c*) Calculate the stress in the beams under the 12 tons point load.

(*d*) Draw ½-inch scale section and side elevation of the beams. R.I.B.A. 1939.

284. A reinforced concrete cantilever projecting 5 feet in the clear is formed by continuing a floor beam over the top of a

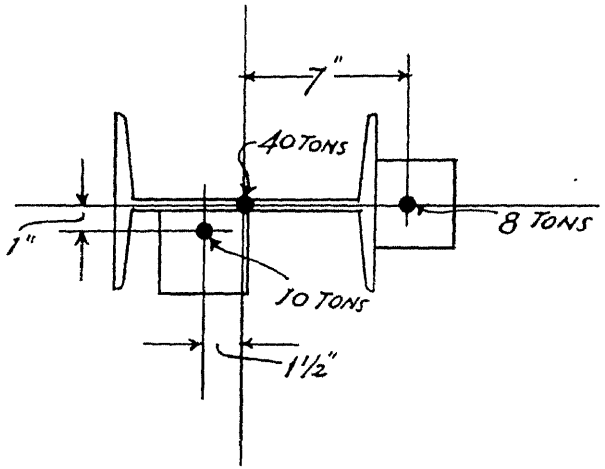


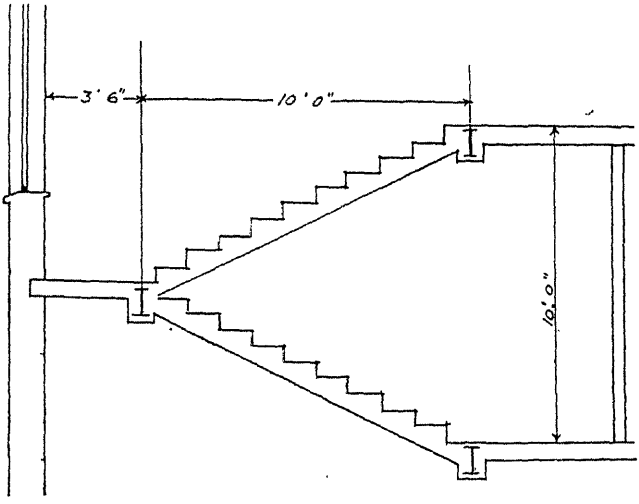
Fig. 12.

reinforced concrete column. The cantilever carries a uniformly distributed load of 4 tons including its own weight and it is 12 inches wide.

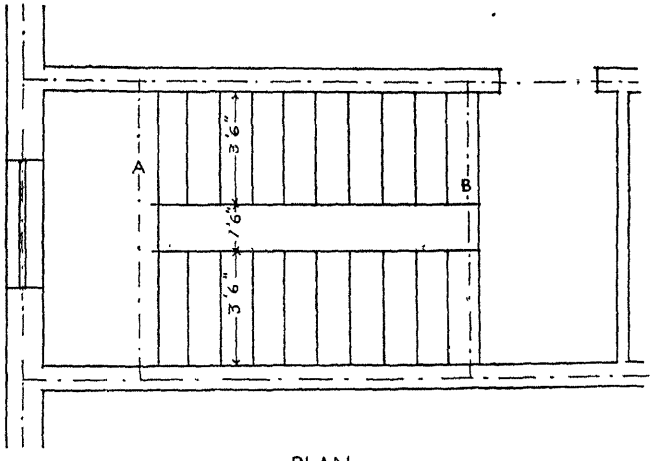
Calculate the dimensions and necessary reinforcement and give explanatory sketches to 1-inch scale showing the position of the rods. R.I.B.A. 1939.

285. A stanchion 12″ × 8″ × 65 lbs. carries an axial load of 40 tons and in addition a flange bracket load of 8 tons and a web bracket load of 10 tons placed as shown in Fig. 12.

(*a*) Calculate the maximum stress in the stanchion.



SECTION



PLAN

Fig. 13.

(*b*) If the height of the stanchion is 16 feet and it is properly restrained in position and direction at each end (both ends fixed) show by any method you know whether or not this stanchion is safely loaded. R.I.B.A. 1939.

286. Define and illustrate with sketches where necessary the meaning of each of the following terms :—

(*a*) Slenderness ratio.
(*b*) Lateral restraint.
(*c*) Distribution reinforcement.
(*d*) Moment of inertia.
(*e*) Radius of gyration. R.I.B.A. 1939.

287. Design a reinforced concrete staircase for the requirements shown in Fig. 13. The superimposed load is 100 lbs. per square foot. Give all calculations and draw a plan and section of the flights to ½-inch scale showing all reinforcement. The staircase is carried on steel beam trimmers A and B shown in Fig. 13. Use any formulæ with which you are conversant and write them out above your calculations. R.I.B.A. 1939.

288. A reinforced concrete column is 18″ × 18″ overall size and carries an axial load of 110 tons. The column is 16 feet high so that no buckling is considered and the full stress of 600 lbs. (direct compression) on concrete may be used.

The modular ratio " m " is 15.

(*a*) Calculate the number of 1-inch diameter vertical rods required.

(*b*) Draw to 1-inch scale plan and part elevation of the column to show all reinforcement.

(*c*) Calculate the shortening of the column under its load.

E for steel ... 30,000,000 lbs. per square inch
E for concrete 2,000,000 lbs. per square inch

R.I.B.A. 1939.

289. A mass concrete vertical wall is 16 feet high and 2 feet thick and stands on a concrete base level with the ground. There is a weak mortar joint between the wall and the base so that no tension stress can be taken by the bed joint. If the maximum wind pressure on the vertical face of the wall is taken as 30 lbs. per square foot, show by calculation whether the wall is safe against overturning. Calculate or find graphically the point along the base of the wall where the resultant pressure acts. The weight of concrete may be taken as 150 lbs. per cubic foot. R.I.B.A. 1940.

290. A timber beam 9 inches wide carries a 9-inch brick wall 11 feet high over a span of 12 feet. Calculate the depth of the beam given that the extreme fibre stress must not exceed 1,200 lbs. per square inch. Brickwork may be taken as weighing 120 lbs. per cubic foot. No triangular loading of the brickwork is to be assumed. R.I.B.A. 1940.

291. A short concrete pier 15″ × 20″ in plan and 8 feet high carries a load of 60,000 lbs, placed eccentrically as shown on the plan given in Fig. 14. Calculate the maximum and

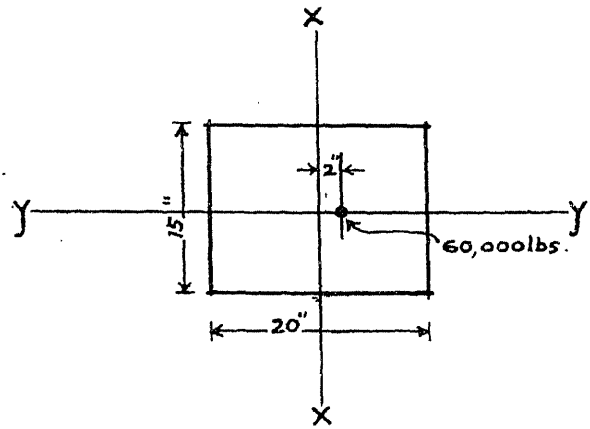


Fig. 14.

minimum stresses on the base in lbs. per square inch. The weight of the column itself may be taken as 2,400 lbs.
R.I.B.A. 1940.

292. A frame diagram is given in Fig. 15 of a steel cantilever truss adequately supported on the line A–B. The loads are indicated in cwts. The principal rafter is divided into three equal bays.

(*a*) Draw the frame diagram to ½-inch scale.
(*b*) Draw the stress diagram to scale of ½-inch equals 10 cwts.
(*c*) Figure on the frame diagram close to each member the stress it receives in tons and indicate tension by a minus sign and compression by a plus sign.
R.I.B.A. 1940.

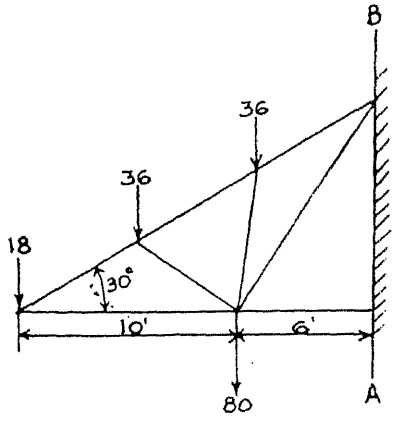


Fig. 15.

293. A steel beam is supported at A and B as shown in Fig. 16 and carries the loads indicated.

(*a*) Draw the shear force diagram.
(*b*) Calculate the maximum bending moment.
(*c*) Calculate the necessary section modulus given that the maximum safe stress is 8 tons per square inch.

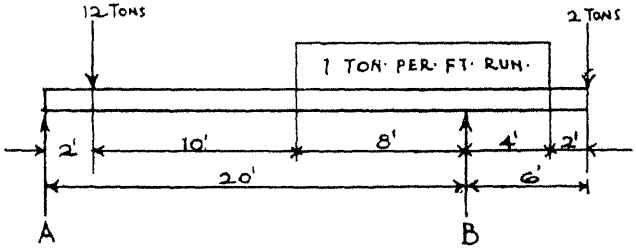


Fig. 16.

(*d*) Find the stress in the extreme fibres of the beam at a point 12 feet from A.

Note.—Do not give the size of steel joist required.

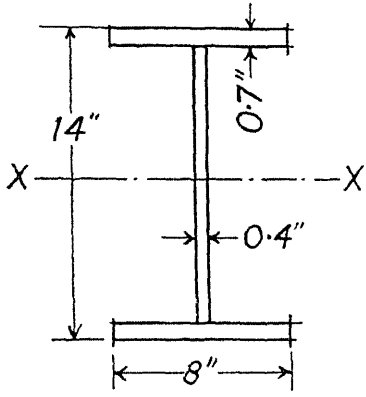
R.I.B.A. 1940.

294. The lower courses of a cylindrical pier in a mediæval church have become crushed and it has been decided to remove them and rebuild the lower part of the pier. The piers, spaced

at 12 feet centres, are about 2 ft. 6 in. in diameter and 10 feet high to the underside of the capitals. The upper part of the pier, the capital and the arches which it supports are all sound.

Make drawings to ½-inch scale with explanatory sketches to a larger scale showing how you would support the sound part of the structure so that the defective parts can be removed and rebuilt. R.I.B.A. 1940.

295. Calculate the approximate moment of inertia, section modulus and radius of gyration all about the X–X axis for the steel section shown. Average web thickness is ·4 inch and average flange thickness ·7 inch.



Calculate also the safe point load that may be put upon a beam of this section if it has a clear span of 20 feet simply supported and the load be placed 4 feet from the support. Safe stress in the extreme fibres may be taken as 8 tons per square inch. R.I.B.A. 1940.

296. A reinforced concrete column is 16″ × 16″ × 10′ high. It transmits an axial load of 220,000 lbs. to a reinforced concrete square base. The soil can carry 2·4 tons per square foot and the concrete base may be taken as weighing 6,500 lbs.

(*a*) Design the column using the minimum bindings (links). The direct stress in the concrete is not to exceed 600 lbs. per square inch and the modular ratio is to be taken as 15. The whole area of the concrete may be regarded as subject to stress. Sketch to 1-inch scale plan and part elevation of the column to show the placing of the steel reinforcement.

(*b*) Calculate a suitable square reinforced concrete base for this column. The compressive stress in the concrete is not to exceed 750 lbs. per square inch, and the tensile stress in the steel is not to exceed 18,000 lbs. per square inch. The modular ratio is to be taken as 15.*

* NOTE.—*The candidate may use the stresses, etc., with which he is familiar in place of those given above but must state fully the basis of his design formulæ.*

R.I.B.A. 1940.

297. A plated girder which is cased in concrete and has its top flange in a concrete floor (adequately restrained) carries the point loads shown in Fig. 17. The total weight of the cased beam is 16 tons. The web plate is 56 inches deep by $\frac{3}{8}$ inch

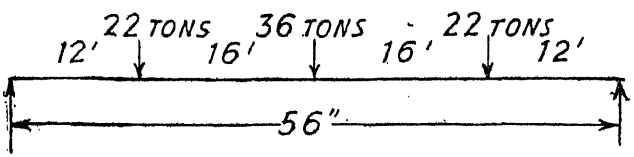


Fig. 17.

thick and the flange plates are to be connected to the web by angles.

Calculate and select suitable flange plates and draw to $\frac{1}{4}$-inch scale the elevation of the girder and a cross section to 1-inch scale with all sizes figured.

R.I.B.A. 1940.

298. One span of a continuous reinforced concrete beam measures 18 ft. 6 in. between brick supporting piers each 18 inches wide on face. The wall above imposes a uniformly distributed load of 1 ton per foot run, and a secondary beam at mid-span imposes a point load of 6 tons. The adjoining bays are of equal span and similarly loaded.

The beam is to be 13 inches wide, the safe stresses in concrete and steel are 750 lbs. and 180,000 lbs. per square inch respectively, and the modular ratio is to be taken at 15. Calculate the depth of the beam and the necessary reinforcement using the economic percentage of steel.

Sketch elevation of part of beam to $\frac{1}{2}$-inch scale showing the arrangement of reinforcement at mid-span and over one of the supports. R.I.B.A. 1940.

299. A small modern house has two living rooms, cloakroom, kitchen, two W.C.s and garage on the ground floor; four bedrooms, bathroom, W.C. and linen cupboard on the first floor, with a flat roof over. There is a water main in the road 30 feet in front of the house. Explain and sketch the cold water supply system, show runs of pipes, give sizes, describe materials, and any precautions against frost. R.I.B.A. 1940.

300. Describe the several materials in general use in the construction of the various types of hollow fire-resisting floors; state the function of each material in each type of floor. R.I.B.A. 1940.

301. Specify all necessary leadwork to a stone-faced chimney stack carried up through a roof, in a position midway between the eaves and ridge. R.I.B.A. 1940.

302. Specify an external staircase and landing in wood, from ground level to the first floor of a workshop, the staircase to be of the step-ladder type, without risers. R.I.B.A. 1940.

303. Describe fully one good quality building stone, giving particulars of quarry, bed, texture, colour and weathering properties. R.I.B.A. 1940.

304. A wood beam, freely supported on a span of 16 feet, is required to carry 3 point loads, each of 8 cwts., placed respectively 4 feet, 8 feet and 12 feet from one support in addition to its own weight, which is 3 cwts. The builder proposes to use two wood joists each 9 inches deep and 3 inches wide securely bolted together.

Do you consider this would be adequate? If not state your recommendations. The maximum stress should not exceed 7 cwts. per square inch. R.I.B.A. 1941.

305. Figs. 18 and 19 show alternative loadings for a steel beam cantilevered at one end and with reactions at A and B.

Calculate the reactions in tons, draw the shear force diagrams to scale, and find the position and amount of the maximum bending moment in each case. R.I.B.A. 1941.

306. Fig. 20 is the frame diagram of a steel truss supported at A and B with loads indicated in hundredweights.

(*a*) Draw the frame diagram to scale.
(*b*) Draw the stress diagram to scale.
(*c*) Figure on the frame diagram the stress each member receives in hundredweights, indicating compression by the plus sign and tension by the minus sign.
R.I.B.A. 1941.

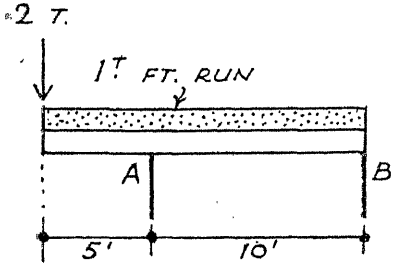


Fig. 18.

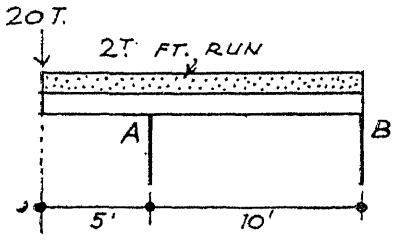


Fig. 19.

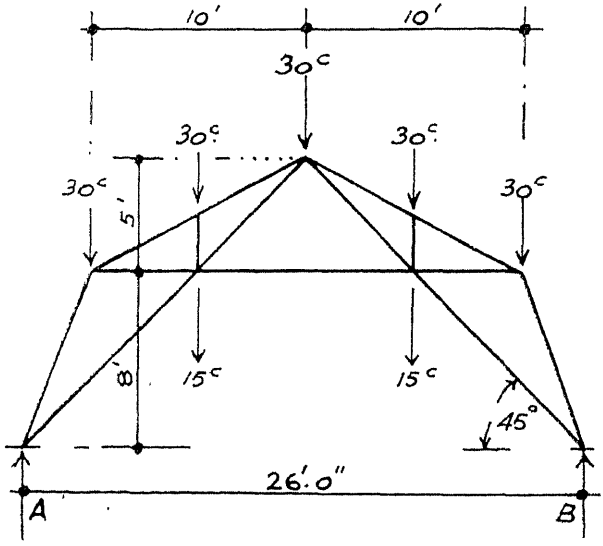


Fig. 20

307. Referring to the roof truss diagram, Fig. 20, describe and explain by careful sketches how you would connect the four members at the apex of the truss. No calculations are required. R.I.B.A. 1941.

308. Draw Fig. 21 to scale of $\frac{1}{2}$-inch equals 1 foot. Then calculate or find graphically the position of the centre of gravity and figure the dimensions to the centre of gravity so that it may be located. R.I.B.A. 1941.

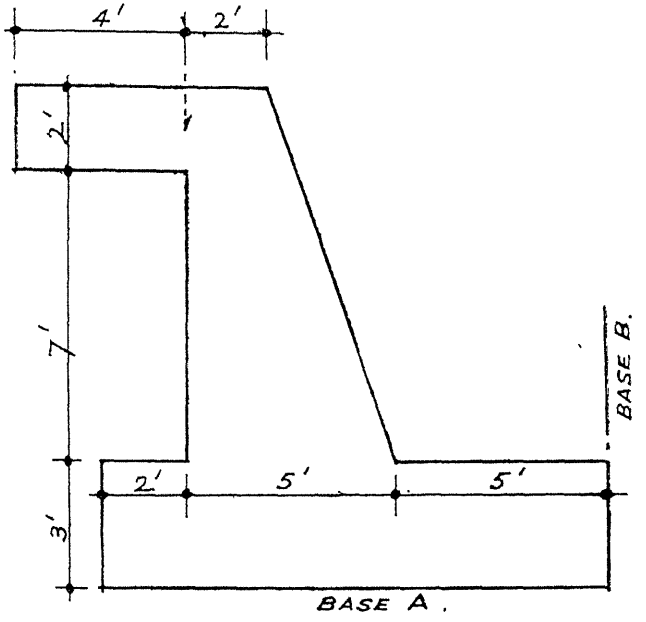


Fig. 21

309. Two 3" × $\frac{1}{2}$" steel flat bars are riveted together to make a single lap joint in a tension member. Three $\frac{3}{4}$-inch rivets in line at $2\frac{1}{2}$-inch centres are used.

Calculate the strength of the joint assuming the following stresses :—

Tension stress not exceeding 8 tons per square inch.
Single shear not exceeding 6 tons per square inch.
Bearing stress not exceeding 12 tons per square inch.

R.I.B.A. 1941.

310. Define briefly but clearly each of the following, using examples and sketches where necessary :—

(*a*) Centre of gravity.
(*b*) Stress and strain.
(*c*) Elasticity. R.I.B.A. 1941.

311. The bed of a stream (at present dry) is 10 feet wide, with banks sloping at 45 degrees to level ground 10 feet above the bed. The subsoil is clay.

A covered footbridge 9 feet wide is to be built in timber across the stream, leaving the bed unobstructed throughout its width to a clear height of 11 feet.

Show complete construction by drawings to ¼-inch scale and one-eighth full size. R.I.B.A. 1941.

312. A glazed verandah is to be added along the south side of an existing building. The verandah is to be 20 feet long and 10 feet wide, approached internally by a door 5 feet wide and 8 feet high. To a scale of ¼ inch to 1 foot make appropriate working drawings.

313. A temporary building is to consist of a range of offices each 15 feet square approached off a corridor 4 feet wide.

Concrete, light steel sections, corrugated steel, and building board are available, but other materials are practically unobtainable. The subsoil is gravel.

Illustrate the complete construction of one unit by plan, elevation and cross section all to ½-inch scale and by careful full-size details. R.I.B.A. 1941.

314. An assembly shop 100 feet long by 50 feet wide has steel stanchions at 12 ft. 6 in. centres along the long side supporting steel trusses spanning 50 feet, having equal pitches of 22½ degrees. The height from floor to underside of truss is 20 feet with a horizontal tie. One gable end is to be of temporary construction to allow for future extension and is to be covered with corrugated iron for its full height on suitable temporary steel framing. In the centre of this gable end sliding doors are required, also of corrugated iron on steel framing, to give a nett opening of 15 feet wide by 12 feet high.

Draw a section and half the elevation of the gable end to ½-inch scale showing clearly all steelwork involved. No calculations are required. R.I.B.A. 1941.

315. A rectangular reinforced concrete beam, 10 inches wide, is supported at A and B and carries an evenly distributed permanent load of 36,000 lbs., including its own weight, as shown in Fig. 22.

Draw the shear force diagram and calculate the necessary bending moments and design the beam. Draw an elevation of the beam to ½-inch scale showing the reinforcement. Use any formulæ with which you are familiar. R.I.B.A. 1941.

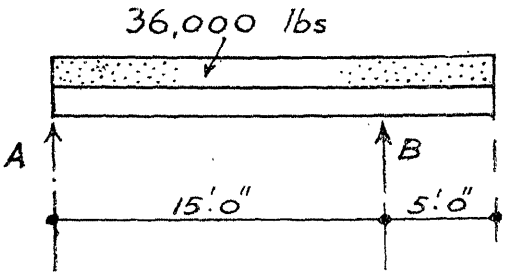


Fig. 22.

316. A square concrete column, 16 feet high, is reinforced with eight 1-inch diameter rods, and carries an axial load of 246,400 lbs. The whole area of the column may be considered effective and the full stress of 600 lbs. per square inch (direct compression) may be used. The modular ratio is 15.

(*a*) Calculate the over-all size of the column.
(*b*) Calculate the shortening under the load.
(*c*) Sketch to 1-inch scale a plan and part elevation to show the reinforcement. R.I.B.A. 1941.

317. A brick pier, 3′ × 2′ 3″ on plan, transmits a load of 68 tons to a square reinforced concrete foundation, which weighs an additional 5½ tons. The soil will safely support 1½ tons per square foot.

(*a*) Calculate for and design the foundation block.
(*b*) Prepare a dimensioned isometric sketch to show the reinforcement. R.I.B.A. 1941.

318. A steel stanchion, 12″ × 8″ × 65 lbs. carries a load of 20 tons central with and 2 inches away from the face of one flange.

(*a*) What additional axial load may be added so that the maximum stress in the stanchion equals 4·5 tons per square inch ?
(*b*) Calculate from the properties in the section book the moment of inertia on the *xx* axis. R.I.B.A. 1941.

319. Describe carefully with annotated sketches the principles and main differences underlying the design of a present-day framed structure in steelwork and in reinforced concrete.

Discuss the advantages and disadvantages of each form of construction.

State, with reasons, your own choice of construction for a transport garage 120 feet clear span and 250 feet long.

R.I.B.A. 1941.

320. Two stanchions, 12 feet apart, transmit loads of 150 tons and 210 tons to a combined grillage consisting of three steel joists cased in concrete (see Fig. 23). The load from the joists is to be evenly distributed on a reinforced concrete foundation of suitable width.

Find the length of the three grillage joists, ignoring their own

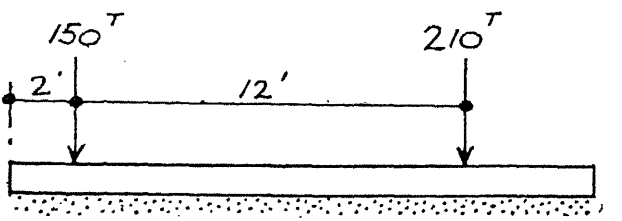


Fig. 23.

weight, and calculate for and select a suitable section assuming a safe stress of 12 tons per square inch. R.I.B.A. 1941.

321. A shop forming part of a row in a crowded urban district is 18 feet wide and 60 feet deep, and the ground floor and basement cover the whole site. There are three upper floors covering the front half of the site only. In the basement there are two w.c.'s, on the first floor there is one w.c. and a sink, on the second floor there is a bathroom and a w.c. and on the third floor there is another bathroom. There is a deep sewer in the road; illustrate the drainage system by sketches. R.I.B.A. 1941.

322. Illustrate by sketches and explain the cold water supply system to a town house three storeys high with a pitched roof over, and containing normal living rooms, kitchens and five bedrooms, assuming that there is a water main in the road.

R.I.B.A. 1941.

323. Discuss the advantages and disadvantages of cement mortar, gauged mortar, and lime mortar for the external walls of a house and give a detailed specification for each type of mortar. R.I.B.A. 1941.

324. A new hospital is to have a flat roof. What are the chief points to consider in choosing the materials for this type of roof and covering? Describe one type of roof and covering which you would use and state reasons for your choice. R.I.B.A. 1941.

325. It is required to construct a flight of steps in the open, leading down to a cellar; the steps are to be capable of withstanding very heavy wear, and provision is to be made for getting rid of the rainwater falling on them. Write a specification of the work, assuming that access to an existing drain is available. R.I.B.A. 1941.

326. Name and describe the various materials used to form a base for plaster on wood-joisted ceilings, and state any precautions necessary in their use. R.I.B.A. 1941.

327. Bending tests carried out on specimen beams 1½ inches wide and 2 inches deep on a span of 20 inches, with a centre point load, gave a safe value for this load of 2 cwts. Using the data available from these test results, calculate the safe load that can be placed on a beam of similar wood, 6 inches deep and 2 inches wide, spanning 12 feet, the load to be uniformly distributed from one support to the mid point of the beam span, *i.e.* load only on one half of beam span. R.I.B.A. 1942.

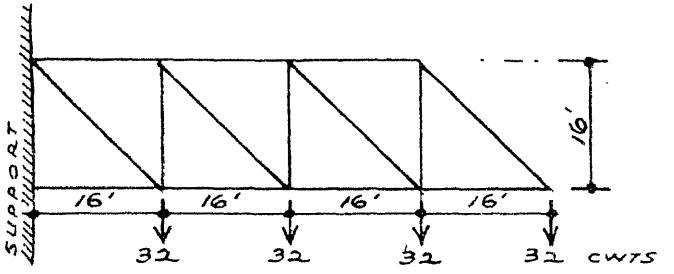


Fig. 24.

328. Draw the cantilever frame shown in Fig. 24 to ⅛-inch scale, then find graphically or by calculation the amount and nature of the stress in each member, and mark each value close

to the respective member in the frame diagram indicating tension by a minus sign and compression by a plus sign. Where graphic solution is used draw the stress diagram to the scale of 16 cwts. to 1 inch. R.I.B.A. 1942.

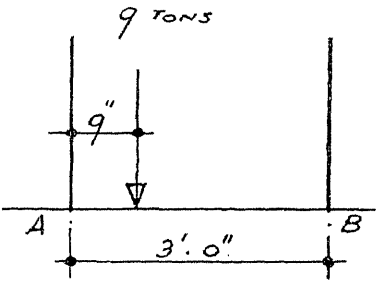


Fig. 25.

329. The resultant pressure at the base of a wall 3 feet thick is given as 9 tons per foot length of wall. The position of the resultant is shown in Fig. 25. Calculate the stress intensities in tons per square foot at points A and B.
R.I.B.A. 1942.

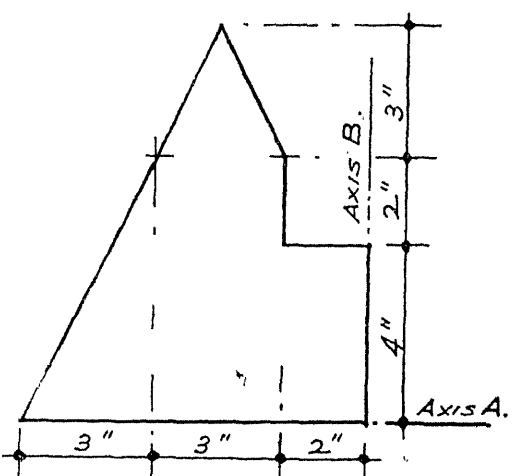


Fig. 26.

330. Draw Fig. 26 half full size and find graphically, or by calculation, the position of the centre of gravity.
R.I.B.A. 1942.

331. Assuming that you have already calculated the maximum stresses in each member of a steel roof truss state clearly the principles governing the following :—
(*a*) The selection of suitable tension members.
(*b*) The selection of suitable compression members.
(*c*) The design of the connections. R.I.B.A. 1942.

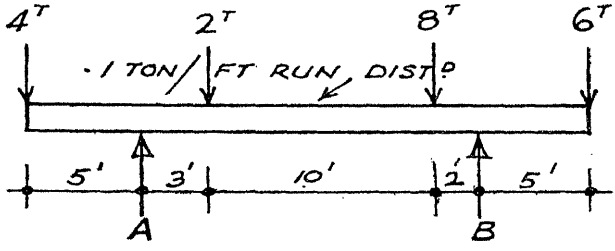


Fig. 27.

332. A steel beam, supported at A and B, carries an evenly distributed load of 1 ton per foot run, including its own weight, in addition to four point loads as indicated in Fig. 27.
Draw the shear force diagram and calculate the position and amount of the maximum bending moment in inch tons.
R.I.B.A. 1942.

333. (*a*) A concrete column having a modulus of elasticity of 2 million lbs. per square inch is 10″ × 12″ and 6 feet high. What concentric compressive load would be required to shorten the column by ·0216 inch ?
(*b*) A circular steel rod 6 feet long having a modulus of elasticity of 30 million lbs. per square inch, elongates ·0216 inch when the same load is applied. What is the diameter of the rod ? R.I.B.A. 1942.

334. A permanent drill hall is to be constructed with a clear span of 40 feet. Draw cross-section together with plan and elevation of one bay to ½-inch scale. Add constructional details one-eighth full size. R.I.B.A. 1942.

335. An underground store is required of internal dimensions 20′ × 10′ × 10′ high. The site is level with clay sub-soil,

and the floor is about 14 feet below ground. Access is required to enable cases up to 2 ft. 6 in. cube in size to be stored. Reasonable ventilation is necessary, and the store must be dry.

Illustrate the construction by plan and sections to ½-inch scale and by details to a larger scale. R.I.B.A. 1942.

336. An open escape staircase is to be built at the end of an existing building to give access to the ground from a floor 30 feet above ground level. The staircase must be completely self-supporting and kept clear of the building. Steel is unobtainable except in mild-steel rods up to ¾-inch diameter.

Show complete construction by means of drawings to ½-inch scale and one-eighth full size. R.I.B.A. 1942.

337. A detached building has brick walls and timber floors and roof. It measures 60′ × 30′ on plan and has windows at 10 feet centres in each wall. The heights are: basement floor to ground level, 5 feet; ground level to ground floor, 4 feet; ground floor to first floor, 12 feet; first floor to second floor, 11 feet; second floor to ceiling under pitched roof, 10 feet.

A bomb, which did not explode, has destroyed the external brick walls at one corner of the building, extending to the first window from the corner in each direction and from the foundations up to the top of the first floor windows.

Show by ¼-inch scale drawings how you would provide temporary support to the structure and describe and illustrate the subsequent permanent repairs you would carry out, detailing the sequence of operations. R.I.B.A. 1942.

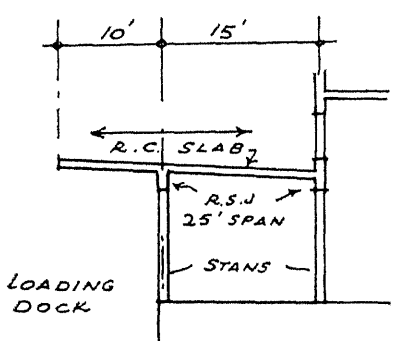


Fig. 28.

338. A loading dock, forming the side of a steel framed warehouse, has a reinforced concrete roof spanning 15 feet and cantilevering a further 10 feet to provide shelter, as shown in Fig. 28. Design the roof slab, assuming an *inclusive* load of 120 lbs. per square foot. and using any formulæ with which you are familiar.

Draw a section of the full length of the slab to ½-inch scale and indicate the reinforcement. R.I.B.A. 1942.

339. Describe in detail the desirable materials, and their proportions, for the concrete in each of the following :—

(*a*) Reinforced concrete slabs in a framed building.

(*b*) Mass concrete in foundations.

Briefly discuss important points of supervision.

R.I.B.A. 1942.

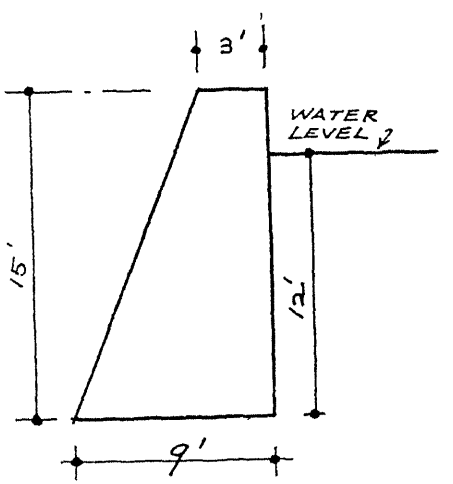


Fig. 29.

340. A gravity retaining dam of masonry, weighing 140 lbs. per cubic foot, is 15 feet high and the vertical face is in contact with the water. The top is 3 feet wide and the base 9 feet wide. The maximum water level height is 12 feet from the base (see Fig. 29). Calculate the maximum and minimum pressures on the base in lbs. per square foot, taking the weight of water as 62·5 lbs. per cubic foot. R.I.B.A. 1942.

341. A 9″ × 7″ × 50 lb. stanchion supports an axial load of 40 tons in addition to the loads of 10 tons and 8 tons on opposite sides of the web, as shown in Fig. 30.

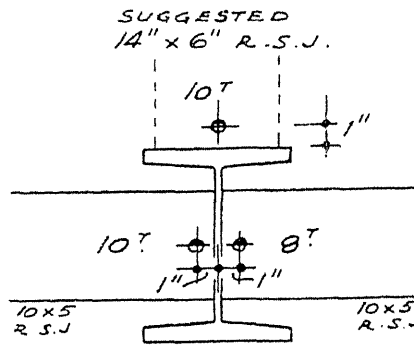


Fig. 30.

It is proposed to add to one flange a 14″ × 6″ R.S.J. having an end reaction of 10 tons. The calculated safe stress for the stanchion is 6 tons per square inch.

(*a*) Make calculations to show whether the stanchion is safe with the added load.

(*b*) Assuming that it is safe, draw a detail of the connection to 1½-inch scale. R.I.B.A. 1942.

342. A 10″ × 10″ compound stanchion transmits a load of 175 tons to a steel slab base 20 inches square. The soil will safely carry 2 tons per square foot. Design a suitable two tier grillage foundation and calculate the thickness of the steel slab base. Assume that the total weight of the grillage foundation is 5 tons. R.I.B.A. 1942.

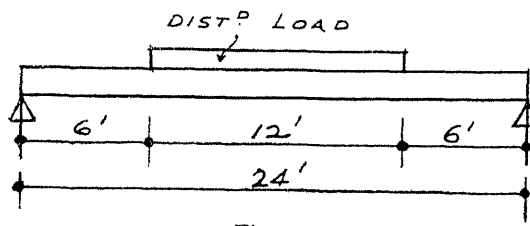


Fig. 31.

343. You are required by a client to utilise an H section steel beam of continental manufacture to support a partially distributed load as shown in Fig. 31. The beam is 12 inches deep and 6 inches wide overall; the average thickness of the flange is $\frac{3}{4}$ inch and the web is $\frac{1}{2}$ inch thick.

(*a*) What distributed load can safely be carried by this beam in the manner indicated, assuming a safe stress of 8 tons per square inch?

(*b*) State the safe uniformly distributed load on a span of 16 feet. R.I.B.A. 1942.

344. The drinking water supply to a new and rather dispersed group of cottages in an agricultural area where the subsoil is chalk, can only be obtained from wells. What initial and permanent precautions would you take to ensure the water being fit to drink? R.I.B.A. 1942.

345. A temporary war-time day-nursery for toddlers only (*i.e.* children 2-5 years of age) requires :—

4 Toddlers' w.c.'s and 4 lavatory basins.
1 Staff w.c.
1 Sink with drinking water.
40 Hooks for towels.
1 Store cupboard.

Plan this lavatory unit, show the drainage and plumbing arrangements, and describe the fittings. R.I.B.A. 1942.

346. What are the points for and against the use for roofing purposes of (*a*) corrugated galvanised iron sheets and (*b*) corrugated compressed asbestos cement sheets? What purlin spacing would you adopt in each case? R.I.B.A. 1942.

347. What are the causes of dry rot? If you found a building with dry rot in one of the ground floor rooms what action would you take? R.I.B.A. 1942.

348. What do you understand by the term Modulus of Elasticity when applied to cast-iron? A cast-iron beam is circular in section and the span is 20 inches. It is found that a central load of 660 lb. causes a deflection of 0·027 inch. If E for cast-iron is 16×10^6 lb. per square inch, under these conditions calculate the diameter of the beam. I. Struct. E. 1939.

349. Specify tests for a mild-steel intended for structual work and describe briefly how these tests would be carried out.
I. Struct. E. 1939.

350. A short composite pillar with flat ends and length 12 in. carries an axial load of 2,000 lb. The pillar consists of a copper tube whose bore is filled by a steel rod. If the area of copper is 0·785 square inch and that of the steel rod is 0·442 square inch, what load is carried by the steel rod ?

E for copper is 16×10^6 lb. per sq. in.
E for steel is 29×10^6 lb. „ „ „
I. Struct. E. 1939.

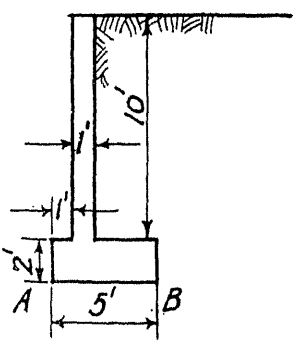


Fig. 32.

351. Fig. 32 shows a cantilever retaining wall of reinforced concrete, supporting filling weighing 100 lb. per cu. ft., and having an angle of repose of 30 degrees. Determine graphically the foundation pressures at A and B if the weight of the concrete is 144 lb. per cubic foot. Do you consider this section to be a satisfactory one ? If not, what alterations do you think advisable ? I. Struct. E. 1939.

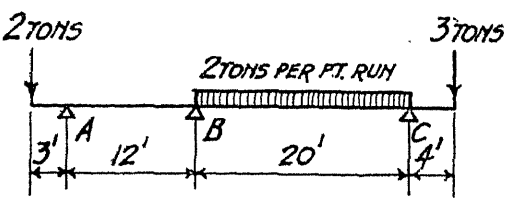


Fig. 33.

352. Draw the bending moment and shearing force diagrams for the beam shown in Fig. 33. I. Struct. E. 1939.

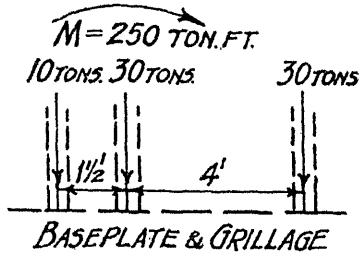
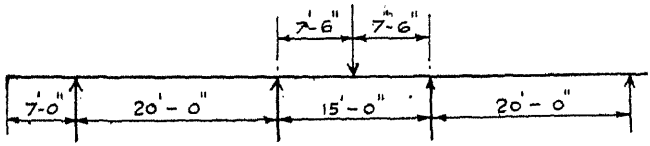


Fig. 34.

353. A stiff grillage foundation supports three legs of a compound stanchion, the legs being spaced and loaded as indicated in Fig. 34. The base of the stanchion is also subjected to an overturning moment (M) of 250 ton-foot due to wind pressure on the side of the building. If the breadth of the foundation is limited to 10 feet, find the length of it so that the pressure on the ground is limited to 3 tons per square foot while one end is at zero pressure. Assume the joists in the lower tier to run lengthwise and find the maximum bending moment on one of the ten joists in this tier.

I. Struct. E. 1939.

354. A reinforced concrete beam continuous over three spans cantilevers beyond one support as illustrated. There is a constant uniformly distributed load of 2,000 lb. per foot run



(including the weight of the beam) throughout the length of the beam and a point load of 8,000 lb. can at times act at the centre of the middle span.

Draw the envelopes of the negative and positive bending moments. Design a suitable rectangular section with reinforcement for the centre span only, and draw a sketch of the elevation and cross section. I. Struct. E. 1939.

Steel in tension	... ...	18,000 lb. per sq. in.
Concrete in compression	...	750 lb. per sq. in.
Concrete in diagonal tension		75 lb. per sq. in.

Value of $m = 15$.

355. A five-storey building, 88′ × 56′ in plan, is to be designed on the flat slab principle for two-way reinforcement.

Two storeys are to be constructed now and three later.

The superimposed floor and roof load is 250 lb. per square foot [illegible] weighing 18 lb. per square foot. The height from floor to floor is 10 feet.

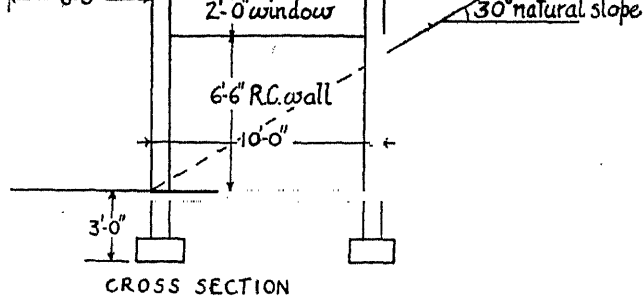
The circular columns are to be at 16 feet centres in both directions, the centres of the outside columns being set in 4 feet from the building line.

Conical caps, 45 degrees slope, are to be 4 feet diameter, with a 2-inch vertical face below a 6 ft. 6 in. square drop panel.

The windows are to be carried on breast walls, 6 inches wide by 3 ft. 6 in. high, with no concrete projection below the slab.

The ground pressure is to be limited to 4 tons per square foot.

The candidate is required to design in detail (1) an internal column base, (2) a corner column from ground to second floor, (3) a corner area of floor, 20′ × 20′ on plan, including cantilevered span and breast wall, and to take out quantities of steel, concrete and shuttering for a corner of the two-storey building 44′ × 28′ in area. I. Struct. E. 1939.



CROSS SECTION

356. A reinforced concrete shelter, 15 feet long overall, is to be erected beside a path at the foot of a long sloping bank, to the dimensions shown on rough section.

The shelter is to be designed for a superimposed load of

50 lb. per square foot on the slab and 30 lb. per square foot on beams and four columns. The reinforced concrete walls are to be designed for an earth thrust, assuming the earth to weigh 120 lb. per cubic foot. The ground pressure is not to exceed 1½ tons per square foot.

The candidate is to design the structure in detail, making clear his proposals for drainage, and take out quantities for steel, concrete, shuttering and excavation. I. Struct. E. 1939.

357. Design and make a detailed sketch of a splice suitable for the pillar section shown in Fig. A. The storey height of

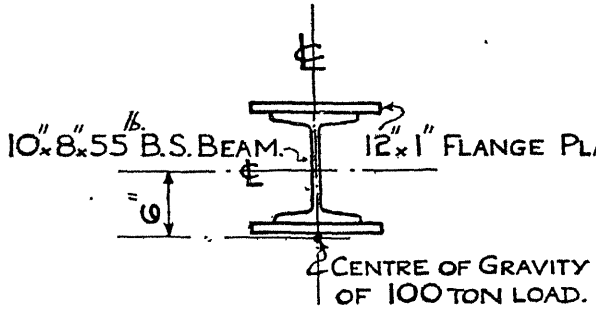


FIG. "A"

the pillar is 20 feet. The load is 100 tons and the centre of gravity of the load is as shown in the sketch.

State whether you consider the pillar is overloaded or not, assuming that heavy four-way connections are provided at the floor levels above and below the storey height of the pillar under consideration.

State the size and pitch of rivets you would adopt for securing the flange plates to the joist shaft.

The following are properties of the section of the 10" × 8" B.S.B., without the flange plates :—

Area	=	16·18 square inches
I_{xx}	=	288·7 inch units.
I_{yy}	=	54·74 „ „ I. Struct. E. 1939.

358. A steel compound stanchion in the basement of a multi-storey building supports a concentric load of 120 tons and is of 9" × 7" × 50 lbs. B.S. beam section with a 10" × ½" plate on each flange.

The stanchion is in an inconvenient position and it is pro-

posed to remove the lowermost 10 feet and to support the remainder on new steel girders.

Design and make a detailed sketch of the new connection between the existing stanchion and the new girders and state the manner in which you would carry out the work.

You may assume a suitable section for the new girders which must be 20 feet span, supported on new stanchions, and arrange them in the most convenient position to simplify the operation. I. Struct. E. 1939.

359. The basement of a six-storey steel-framed office building with reinforced concrete floors must be strengthened to provide an air raid shelter, against collapse of the superstructure.

The existing stanchions are spaced at 20-feet centres in each direction and the existing steel beams are spaced at 10-feet centres in one direction and at 20-feet centres in the other direction, *i.e.*, opposite the stanchions.

The height of the basement from floor to underside of the existing concrete slab is 11 ft. 6 in. and to underside of beam casing 10 feet.

Draw a sketch showing how you would undertake the strengthening of one 20′ × 20′ bay and describe the method of inserting the new materials which must be independent of any support from the existing structure.

The ground below the existing basement is capable of supporting a load of 2 tons per square foot. I. Struct. E. 1939.

360. A freely supported beam ABC 24 feet long is continuous over two spans of 12 feet. The three supports are at the same level and the beam is loaded as shown in Fig. 35.

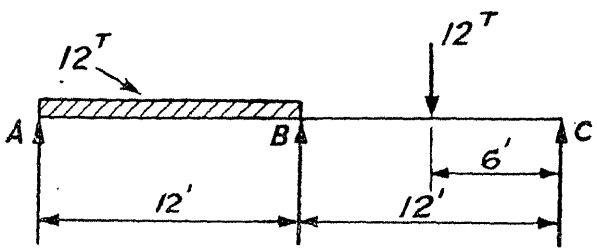


Fig. 35.

Draw the bending moment and shear force diagrams giving all important values, and find reactions at A, B and C.

I. Struct. E. 1941.

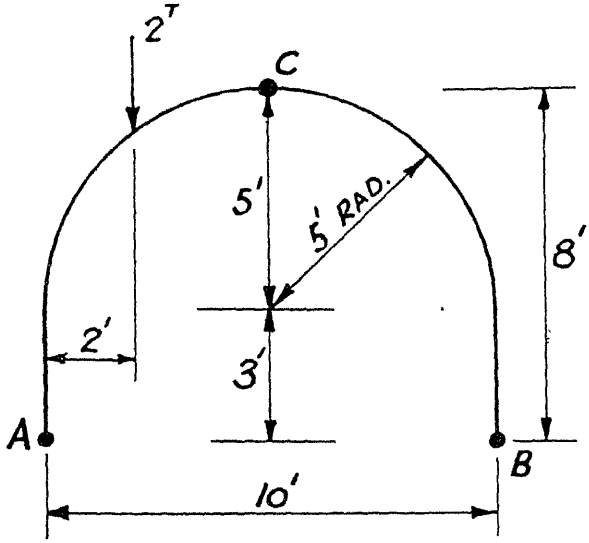


Fig. 36.

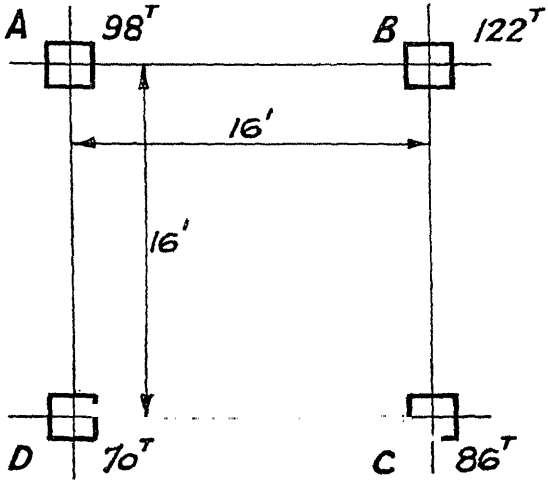


Fig. 37.

361. Fig. 37 shows the plan of, and loads carried by, four columns which are to be supported on a common concrete foundation, square in plan. Neglecting the weight of the concrete foundation itself, calculate the size of the foundation if the ground pressure is uniform and is not to exceed 1 ton per square foot. Give the distances of the centre of the foundation from the centre lines of the stanchions AB and AD.
I. Struct. E. 1941.

362. A freely supported beam, 24-feet span, carries a load of 12 tons uniformly distributed along its whole length, together with a load of 9·8 tons uniformly distributed along a length of 14 feet from the left-hand support.

Find the position and value of the maximum bending moment and draw to scale the shear force diagram giving all important values. I. Struct. E. 1941.

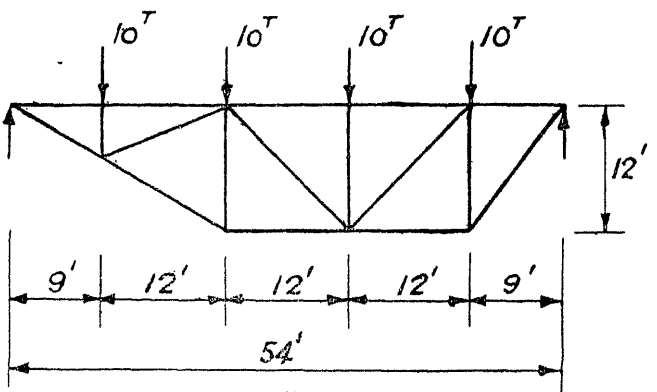


Fig. 38.

363. A braced girder is shown in Fig. 38. Determine graphically or otherwise the forces in all the members due to the loading shown and indicate clearly which members are in tension and which are in compression.

364. A three-hinged semi-circular arch rib ACB, 5 feet radius and with vertical supports, is shown in Fig. 36.

(*a*) Draw the line of pressure due to the 2-ton load in the position indicated.

(*b*) Determine the vertical and horizontal reactions at A and B.

(*c*) State the maximum bending moment and show where it occurs. I. Struct. E. 1941.

365. A flitched beam consists of a timber beam with a wrought iron plate of the same depth fastened to it. It carries a uniformly distributed load of 7 tons over a span of 12 feet. If the breadth of the timber is six-tenths of the depth and that of the flitch plate is one-twelfth that of the timber, find suitable dimensions.

Assume E for timber = 800 tons per square inch.
E for wrought iron = 12,500 tons per square inch.
f for timber = $\frac{1}{2}$ ton per square inch.
f for WI = 6 tons per square inch.

I. Struct. E. 1941.

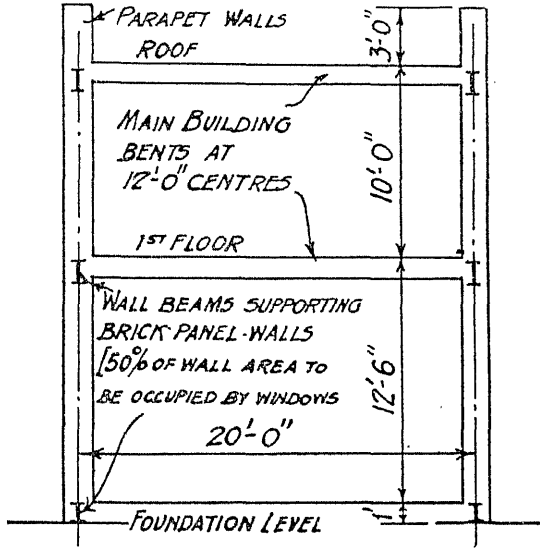


Fig. 39.

366. Assume rigid or semi-rigid frame construction and design and make detail drawings to a scale of $\frac{3}{4}$ inch = 1 ft. 0 in. of the main bents of the structure shown in Fig. 39.

The roof is composed of 5 inches of reinforced concrete, an average of $1\frac{1}{2}$ inches top screed and $\frac{3}{4}$ inch of asphalt, and supports a superimposed evenly distributed load of 50 lbs. per square foot.

The first floor is composed of 8 inches of reinforced concrete and supports a superimposed evenly distributed load of 220 lbs. per square foot.

The roof and first floor are plastered on the underside.

The walls are of 13½-inch brickwork and the steel is encased in concrete with a minimum cover of 2 inches.

Note that a design which assumes freely supported end conditions for the beams cannot be accepted.

I. Struct. E. 1941.

367. A 3-hinged portal type shed building is required, 120 ft. 0 in. long by 60 ft. 0 in. span by 20 ft. 0 in. clear height to eaves.

The roof is to be covered with large corrugated asbestos sheeting and to provide for a suitable amount of roof lighting.

The walls are to be of 9-inch brickwork for a height of 6 ft. 0 in. and the remainder covered with asbestos corrugated sheeting.

Make a drawing to a scale of ⅛ inch = 1 ft. 0 in. showing an economical arrangement of the whole of the steelwork.

Design and make a detailed drawing of one-half of one of the main portal frames to a scale of ¾ inch = 1 ft. 0 in.

I. Struct. E. 1941.

368. A mass concrete retaining wall is 16 feet high, 4 feet wide at the top, 9 feet wide at the base and has a vertical back. For the earth being retained w = 110 lbs. per cubic foot, the angle of repose ϕ = 30 degrees and the "fill" is level with

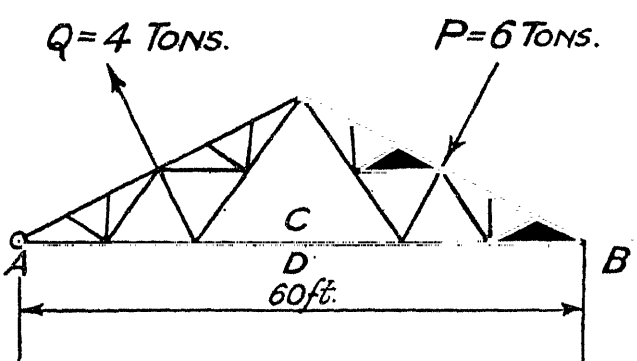


Fig. 40.

the top of the wall. In addition there is a superimposed load acting on the top of the earth equal to 2 cwt. per square foot. Investigate the stability of the wall and draw to scale the diagram of vertical pressure distribution on the soil.

The material of the wall weighs 144 lbs. per cubic foot and the earth pressure intensity at a depth h feet is $wh\left(\frac{1-\sin\phi}{1+\sin\phi}\right)$ lb. per square foot horizontal. I. Struct. E. 1941.

369. A roof principal of the form shown in Fig. 40 has a rise of quarter of the span and is subject to a total wind pressure P on one side and wind suction Q on the other side, acting normal to the rafter slope. If the reaction at B is vertical find the directions and magnitudes of the reactions at A and B and the force in the member CD due to this loading.

I. Struct. E. 1942.

370. A beam of length L feet overall carries a uniformly distributed load w per foot. It is supported at two points distant a feet from either end. Draw the general form of the shear force and bending moment diagrams and show that the maximum bending moment will have its least possible value when $a = \cdot 207$ L. I. Struct. E. 1941.

Concrete in compression ... 750 lbs. per square inch.
Steel in tension ... ... 18,000 lbs. per square inch.
Value of $m = 15$.

371. Describe the manufacture of Portland cement; in what way does it differ from (*a*) rapid hardening Portland cement and (*b*) high alumina cement?

Describe briefly the standard laboratory tests for Portland cement and a method of testing for soundness which can be carried out on the site. I. Struct. E. 1941.

372. State the assumptions upon which reinforced concrete design is based and comment briefly on their validity.

A reinforced concrete column 12 inches square reinforced with four $\frac{3}{4}$-inch diameter bars having 1 inch cover, carries an eccentric load of 20 tons; the eccentricity is such that the bars on one face are unstressed.

Calculate :—

(1) The maximum stress in the concrete.
(2) The stress in the bars on the other face.
(3) The eccentricity of the load. I. Struct. E. 1941.

373. State the regulations for determining the breadth of the flange of a tee beam when used in beam and slab floor

construction. The beam shown in Fig. 41 carries a total load of 2,500 lbs. per foot run on a freely supported span of 20 feet. Calculate the necessary areas of compression and tensile reinforcement. If the compression reinforcement were omitted and twice the previous amount of tensile reinforcement used, what load would the beam then carry on the same span ?

I. Struct. E. 1941.

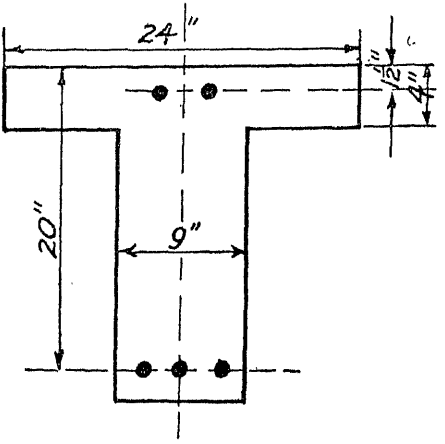


Fig. 41.

374. The continuous beam shown in Fig. 42 is supported at A, B, C, and D and carries loads as shown. If the flexural rigidity is constant throughout, determine the bending moments at the supports and draw the bending moment and shear force diagrams.

If the supporting piers are of brickwork having a safe compressive stress of 140 lbs. per square inch, what sizes should they be made ?

I. Struct. E. 1941.

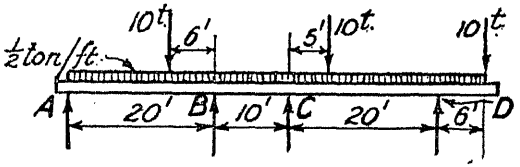


Fig. 42.

375. The wall shown in Fig. 43 retains earth weighing 110 lbs. per cubic foot and having an angle of repose at 30 degrees. The surface, which is horizontal, carries a load of 2 cwt. per square foot.

You are required to calculate :—

(i) The thickness of X–X and the area of steel required at that section.
(ii) The thickness of the base.
(iii) The factor of safety against overturning.
(iv) The distribution of pressure on the base.

In the above cases the friction on the back of the wall can be neglected, but taking the coefficient of friction between the base and the earth to be 0·5 and allowing for any passive resistance of the earth, calculate the factor of safety against sliding, assuming uniform condition of the soil.

Draw the profile of the wall with the thicknesses you have calculated and sketch the main tensile reinforcement which should be arranged as economically as possible.

I. Struct. E. 1941.

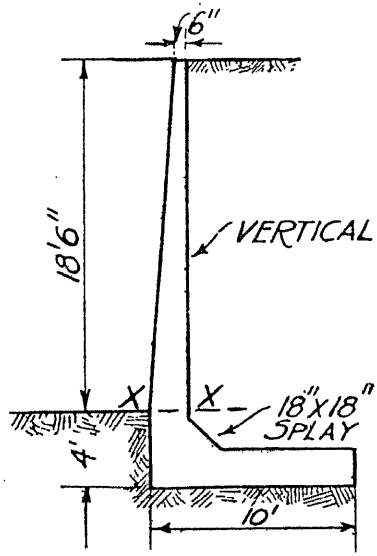


Fig. 43.

376. Design and make a detail drawing to a scale of $\frac{3}{4}$ inch to 1 ft. 0 in of one-half of a steel road bridge suitable for a span of 60 ft. 0 in. between abutments. The bridge must be of the through type and have a clear carriage way of 20 ft. 0 in. with a 7 ft. 6 in. wide footpath in each side and must be suitable for modern heavy traffic conditions. I. Struct. E. 1941.

377. Make an outline drawing of a scheme of construction showing the main steelwork for a modern garage type of building having a clear floor area of 200′ 0″ × 120′ 0″ without internal stanchions.

Design and make a detail drawing of one-half of one of the main girders to a scale of $\frac{1}{2}$ in. = 1 ft. 0 in. I. Struct. E. 1941.

378. A 10″ × 6″ B.S.B. stanchion has an effective length of 9 ft. 0 in. It supports axial loads of 30 tons due to dead loading and 17 tons due to wind pressure. It also resists an overturning moment at the base of 84 tons inches acting about its major axis, this being entirely due to wind pressure. The properties of the section of the stanchion are as follows :—

Area = 11·77 square inches.
Maximum moment of inertia = 204·8 inch units.
Minimum moment of inertia = 21·76 inch units.

Check by calculation whether the stanchion will safely withstand this loading. I. Struct. E. 1941.

379. Many authorities consider that slender members such as flat steel bars should not be used in the steel framing of buildings, particularly in roof trusses. State fully the reasons for and against the adoption of such members and how far you are in agreement with this view. I. Struct. E. 1941.

380. Design and make a detail drawing of a suitable mass concrete foundation and steel base for a 9″ × 7″ B.S.B. stanchion which has an overall height of 12 ft. 0 in. from base to cap, supports an axial load of 20 tons and is subjected to a horizontal pull of 1 ton at the cap. The subsoil is compact gravel. I. Struct. E. 1941.

381. A plate girder is composed of a web plate 48″ × $\frac{1}{2}$″. Four flange angles each 6″ × 6″ × $\frac{5}{8}$″ and two flange plates (one each flange) each 14″ × 1″. There are double angle web stiffeners spaced at 3 ft. 6 in. centres of 4″ × 4″ × $\frac{1}{2}$″ angle section riveted with $\frac{7}{8}$-inch diameter rivets at 4-inch pitch. It is necessary to cut an 18-inch diameter hole through the centre of one of the panels of the web plate to allow a pipe to pass

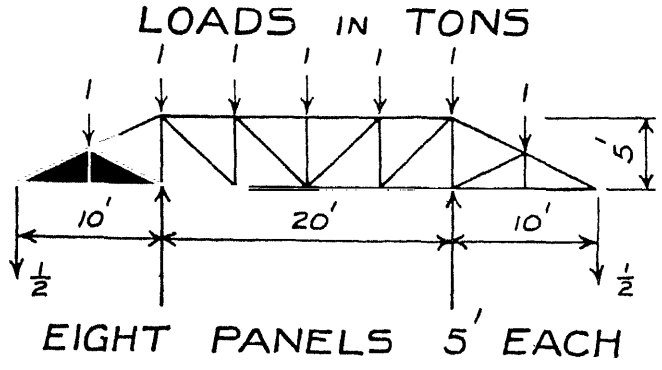

LOADS IN TONS
5'
10'
20'
10'
½
½
EIGHT PANELS 5' EACH

Fig. 44.

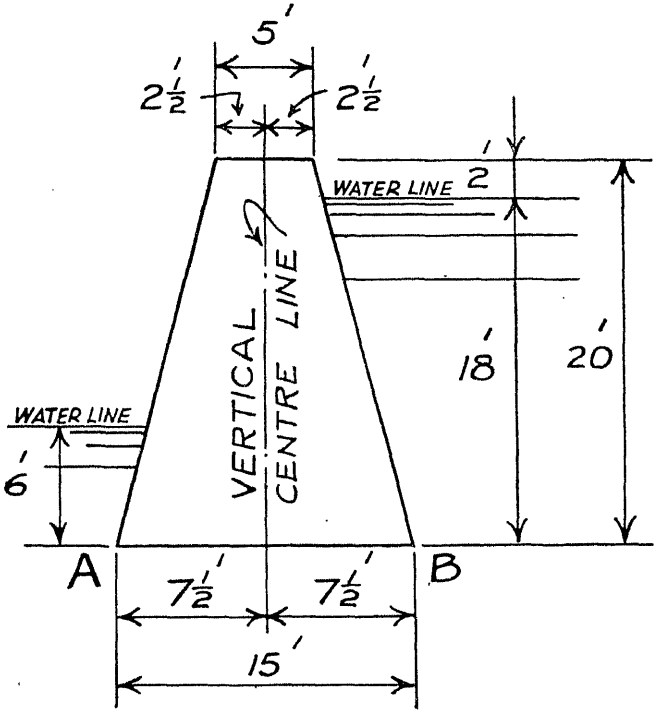

5'
2½'
2½'
WATER LINE
2'
VERTICAL CENTRE LINE
18'
20'
WATER LINE
6'
A
B
7½'
7½'
15'

Fig. 45.

through. The total shear on the girder at the point where the hole has to be made is 90 tons. State whether you consider that additional stiffening should be provided for the web plate, and, if so, make a sketch showing your proposals.

I. Struct. E. 1941.

382. A railway platform roof is of the form shown in Fig. 44. Find either graphically or by calculation, the forces in all the members due to the loading indicated and show clearly which members are in compression and which in tension.

I. Struct. E. 1940.

383. A concrete wall for a reservoir is of trapezoidal section as shown in Fig. 45. It is subject to water pressures due to a depth of 18 feet on the upstream face and due to a depth of 6 feet on the downstream face. If the concrete weighs 150 lb. per cubic foot, find the magnitude of the resultant pressure on the base AB and the position of the centre of pressure. Weight of water = 62·5 lbs. per cubic foot. I. Struct. E. 1940.

384. Fig. 46 shows in plan a beam CED anchored at D and resting upon a second beam AB at E. Beam AB is freely supported at A and B. There is a load of 8 tons at C and a

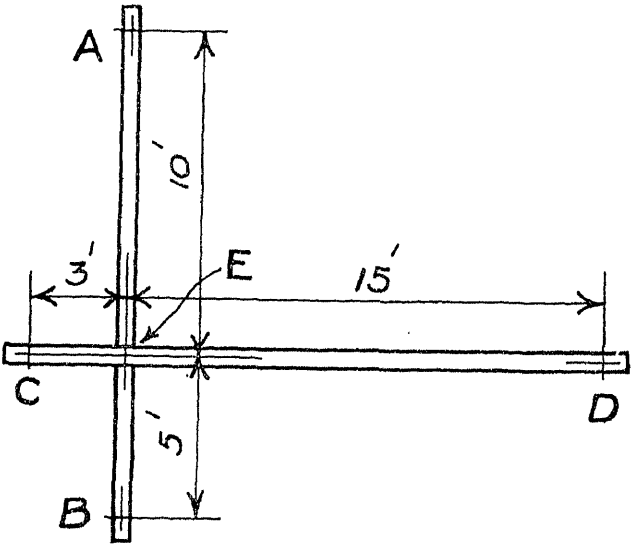


Fig. 46.

total uniform load from E to D of 2 tons. The end D is unsupported. Find (*a*) the reactions at all the supports and (*b*) the necessary modulus of section of the beam AB if the working stress is 8 tons per square inch. I. Struct. E. 1940.

385. A steel bracket as shown in Fig. 47 is used for hoisting purposes. Ignoring any effects of the hoisting rope, determine

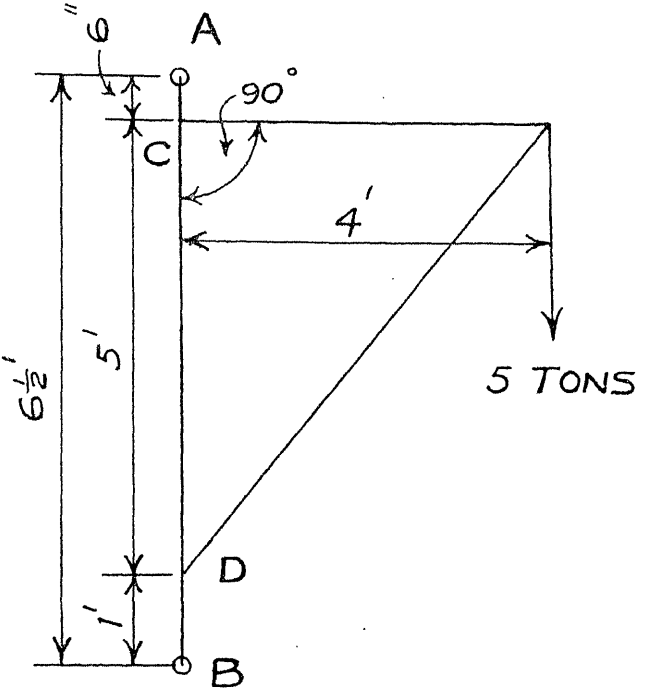


Fig. 47.

the particulars for, and draw the bending moment and shear force diagrams for the supporting post AB, which is hinged at the supports. State the magnitudes of the bending moments at C and D. I. Struct. E. 1940.

386. A beam ABCDE 25 feet overall length is carried on three rigid supports at the same level at B, C and D. There is a load of 2 tons at each end A and E, and a load of 8 tons in the

middle of each span BC and CD. AB = DE = $2\frac{1}{2}$ feet. BC = CD = 10 feet.

(*a*) Calculate the bending moments at B, C and D.
(*b*) Calculate the reactions at B, C and D.
(*c*) Draw the bending moment diagram.
(*d*) Draw the shear force diagram. I. Struct. E. 1940.

387. Explain the terms Limit of Proportionality and Yield Point, illustrating your answer by a sketch of a typical load-extension diagram for mild steel.

The following data were obtained from a tensile test on a mild steel specimen :—

Diameter, $\frac{3}{8}$ inch.
Gauge length, 8 inches.
Extension at load of 1·2 tons, $6{\cdot}5 \times 10^{-3}$ inches.
Yield load, 1·62 tons.
Maximum load, 2·9 tons.
Elongation at fracture, 2·42 inches.

Calculate (*a*) Young's Modulus ; (*b*) the stress at the Yield Point ; (*c*) the Ultimate Stress ; (*d*) the percentage elongation at fracture. How would you find the true breaking stress ?
I. Struct. E. 1940.

388. A rolled steel joist, with ends freely supported on a span of 30 feet, is required to carry a concentrated load of 16 tons at the centre of the span. If the maximum stress due to bending is not to exceed 8 tons per square inch and the central deflection is not to be greater than 1/400 of the span, find a suitable depth and section modulus for the beam. Neglect the weight of the beam itself. E = 13,000 tons per square inch. I. Struct. E. 1940.

389. Describe, with the aid of a sketch, a Cement Testing Machine and state how the briquettes are prepared and tested. What results would you expect from the tensile testing of a modern Portland cement ?

Mention briefly any other tests usually applied to cement.
I. Struct. E. 1940.

390. A short circular concrete column is 12 inches in diameter and carries a vertical load of 12 tons which is applied 2 inches distant from the axis. Obtain a formula to enable you to calculate the maximum and minimum stresses due to the load.

Give a diagram showing how the stress varies across the column section. I. Struct. E. 1940.

391. Fig. 47 shows a section of an existing brick retaining wall with a concrete foundation. A length of this wall is to be removed and to be replaced by a reinforced concrete cantilever retaining wall having approximately the same measure of stability as the existing wall. Assuming that the dimensions of the base slab remain unaltered, determine a suitable thickness of stem and length of heel, and make a dimensioned sketch of the reinforced concrete wall. The earth weighs 100 lb. per cubic foot $\phi = 30$ degrees. Details of reinforcement are not required. I. Struct. E. 1940.

392. A steel column carrying an axial load of 250 tons rests on a bloom base which is supported by a two-tier grillage foundation. The maximum allowable pressure on the foundation is 2 tons per square foot and the bloom base is 2 ft. 6 in. square. Design the grillage whose breadth is restricted to 9 ft. 6 in. I. Struct. E. 1940.

393. A plate girder of small span is composed of a $30'' \times \frac{1}{2}''$ web plate, four $6'' \times 6'' \times \frac{5}{8}''$ flange angles and two $14'' \times \frac{5}{8}''$ flange plates.

The reaction at each reinforced concrete bearing is 60 tons.

Determine exactly the required pitch of $\frac{7}{8}$-inch diameter rivets for securing the flange angles to the web plate.

Design and make a detailed sketch of a suitable bearing and end stiffener for the girder to a scale of $\frac{1}{2}$ inch = 1 ft. 0 in.

Calculate the maximum spacing of the web stiffeners near the end of the girder. I. Struct. E. 1940.

394. A pitched roof of 30 degrees slope is covered with corrugated iron sheeting.

The steel trusses are spaced at 15 ft. 0 in. centres and the steel purlins are of $4'' \times 3'' \times \frac{3}{8}''$ angle section spaced at 6 ft. 0 in. centres.

The purlins are jointed over the trusses and are spliced at their joints which develop 75 per cent. of the full strength of the section.

Determine the maximum tensile and compressive stresses in the purlins.

You may neglect the curvature at the root and toes of the angle section for purposes of calculation. I. Struct. E. 1940.

395. A $16'' \times 8''$ rolled steel joist spans 20 ft. 0 in. centres of freely supported bearings and supports an evenly distributed load of 20 tons, which includes its own weight.

The beam is not supported laterally in any manner.

The load is applied 2 inches eccentrically from the y.y. axis of the beam.

Determine the maximum tensile stress in the beam.

State whether the eccentric application of the load causes a greater deflection than a similar concentric load would cause.

Make a sketch to a scale of $\frac{1}{2}$ inch = 1 ft. 0 in. showing what method you would adopt of preventing excessive torque of the beam.

The properties of a 16″ × 8″ rolled steel joist section, in inch units, are as follows :—

Area 22 square inches.
Moment of Inertia, xx = 974.
Moment of Inertia, yy = 68. I. Struct. E. 1940.

396. An elevated coal bunker is 15 feet wide and 30 feet long, divided by a central wall into two equal pockets.

The depth to the top of hopper bottoms is 15 feet, and the further depth to the two hopper mouths is 8 feet. The mouths are 18 inches square and left flat for the attachment of steel outlet and gear weighing 1 ton. The feeding conveyor (discharging 5 feet above top of bunker) and superstructure 10 feet high brings point loads of 4 tons on each corner and 6 tons on the centre of both long sides.

The bunker is to be supported on six columns of sufficient length to give 16 feet height between ground level and bottom of hopper concrete. The foundations are to be taken 4 feet below ground where a safe pressure of $2\frac{1}{2}$ tons per square foot may be assumed.

The coal weighs 56 lbs. per cubic foot and has an angle of repose of 40 degrees. Wind pressure at 15 lbs. per square foot is to be taken into account on the vertical bunker sides and superstructure.

(*a*) Prepare $\frac{1}{8}$ inch scale plan and elevations.
(*b*) Detail to $\frac{1}{2}$ inch or 1 inch scale the bunker and hopper bottoms and at least the internal columns.
(*c*) Prepare bill of quantities for shuttering and concrete for the hopper bottoms and walls.
Prepare bar bending list for the hopper bottoms.
I. Struct. E. 1940.

Steel in tension	8 tons per square inch.
Concrete in compression	750 lbs. per square inch.
Concrete in diagonal tension	75 lbs. per square inch.

Detail drawings for both questions must be to a scale of not less than $\frac{3}{4}$ inch = 1 foot.

397. Design and make a detailed drawing of one-half of a girder spanning 80 ft. 0 in. centres of blue brick supports.

The girder supports an evenly distributed load of 120 tons and a point load of 150 tons situated 30 feet from the centre of one of the supports.

The girder may be assumed to be braced laterally to another girder of similar dimensions.

The drawing must be suitable for workshop use.

I. Struct. E. 1940.

398. It is proposed to provide steel sheet piling for the excavation for a foundation block which is 70 feet long by 20 feet wide and which has to find a bottom 2 feet into blue clay.

The strata of the subsoil is as follows :—

6 feet top soil and made ground ;
4 feet loam ;
2 feet sand ;
Ground water level (constant) ;
12 feet sandy ballast ;
1 foot sand ;
3 feet ballast ;
2 feet light brown clay
4 feet mottled clay ;
Blue clay level.

Design the steel sheet piling and draw a detailed transverse section through the excavation to a scale of $\frac{3}{4}$ inch = 1 foot and draw a plan to a scale of $\frac{1}{8}$ inch = 1 foot.

State the procedure of the work.

You may assume a section for the steel sheet piling and make approximate calculations for the sectional properties.

Calculations must also be made for the sections required.

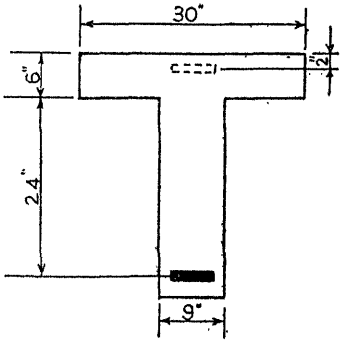


Fig. 48.

399. A Tee beam as in Fig. 48 is reinforced in tension so that when loaded the maximum stresses in steel and concrete are as given above.

Calculate from first principles :—

(*a*) The resistance moment and area of tensile reinforcement with no compression reinforcement.

(*b*) The resistance moment and area of tensile reinforcement with 3 square inches of compression reinforcement in the top with 2-inch embedment.

(*c*) The equivalent moment of inertia about the neutral axis in each case. I. Struct. E. 1940.

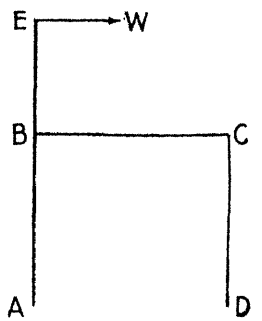


Fig. 49.

400. A portal ABCD in Fig. 49 has one vertical AB extended to E at which a horizontal load W is applied.

Draw deflection and bending moment diagrams :—

(*a*) If A and D are hinged and B and C are constrained to remain vertically above A and D respectively.

(*b*) Due to the removal of the restraint at B and C.

(*c*) As for (*a*) but with A and D fixed.

(*d*) As for (*b*) but with A and D fixed.

If (i) AB, BC and CD are of equal section and length, (ii) A and D are hinged and (iii) there is no side restraint, calculate the moments in the beams and the columns at B and C due to a 1-ton horizontal load at E 10 feet above BC.

I. Struct. E. 1940.

401. A 20″ × 12″ column reinforced on each 12-inch face with four 1-inch round bars with $1\frac{1}{2}$-inch cover carries a load of 130,000 lbs. together with a bending moment at the top of the base of 480,000 lbs. inch in either direction about the axis parallel to the shorter side.

The shear in the column at the top of the base is 5,000 lb.

Calculate the stresses in the concrete and steel of the column and design a base 24 inches deep and 8 ft. o in. long parallel to the 20-inch face, so as to limit the ground pressure to $1\frac{1}{2}$ tons per square foot exclusive of the weight of the base and the earth above it. I. Struct. E. 1940.

402. Fig. 50 shows the section of a bridge pier of concrete with the forces acting per foot run. Assuming that the central load of 7·3 tons includes the weight of the pier itself, find

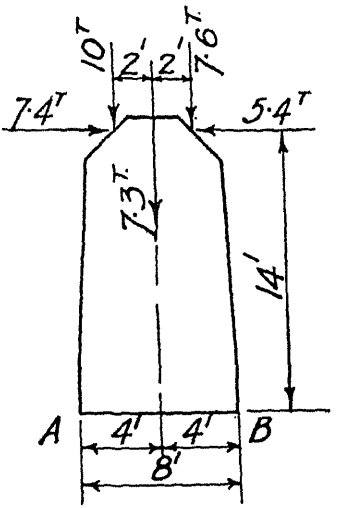


Fig. 50.

(*a*) The centre of pressure on the base AB.

(*b*) The vertical pressure distribution on the base AB. I. Struct. E. 1940.

403. Fig. 51 shows half of a segmental rib supporting a series of vertical loads at 5-feet intervals. The arch is three-hinged and the loading is symmetrical on each side of the centre line CL.

By GRAPHICAL construction determine: (1) the magnitude and position of the resultant load on the half arch; (2) the moment of this load about the support A; (3) the horizontal thrust T required to support the loaded rib. I. Struct. E. 1940.

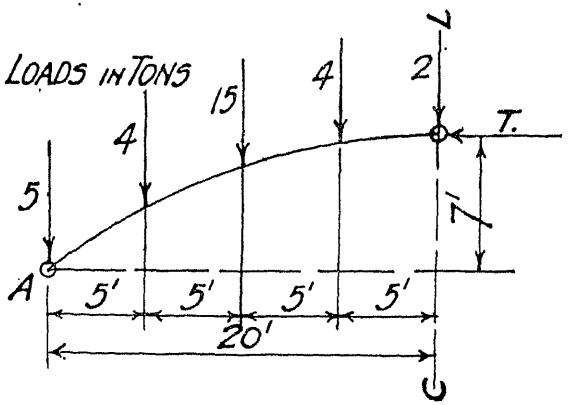


Fig. 51.

404. Fig. 52 shows a steel vertical member 20 feet high, with pinned ends, subject to a uniform horizontal load of $\frac{1}{2}$ ton per foot run of height and two point loads of 4 tons, one on each side, forming a couple. Determine the reactions at A and B,

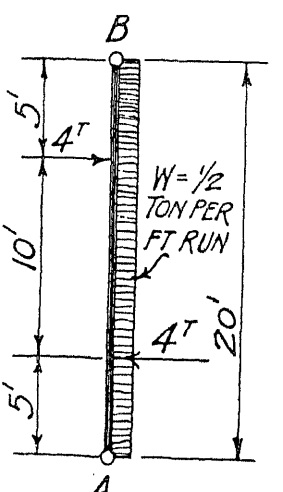


Fig. 52.

the position of zero shear and the value of the maximum bending moment.

Draw the shear force diagram for the member and calculate the modulus of section required for such a member at a working stress of 8 tons per square inch. I. Struct. E. 1940.

405. The standard connection for a 16″ × 6″ × 62 lb. B.S. beam is shown in Fig. 53. Determine the safe end reaction on such a beam connection, assuming that the reaction acts at the back of the angle cleat, along the line YY. Use $\frac{3}{4}$-inch rivets. The web of the beam is 0·55 inch thick.

I. Struct. E. 1940.

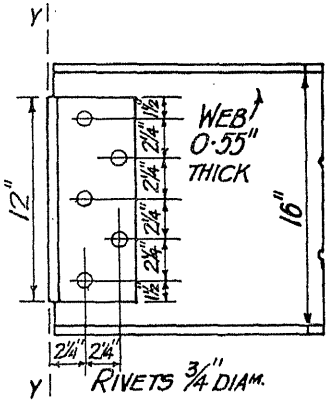


Fig. 53.

406. A beam of reinforced concrete has equal reinforcement in the tension and compression areas as shown in Fig. 54. Assume the allowable stress in the steel to be 18,000 lb. per square inch; the allowable compression in the concrete to be 750 lb. per square inch; and the modular ratio to be 15. Determine the moment of resistance.

(i) taking compressional concrete into account,
and (ii) on the resistance of the steel alone.

In (i) give the actual stresses on the steel in tension and compression. Compare the two results and discuss the two methods, giving your views as to the practical expediency of method (ii).

I. Struct. E. 1940.

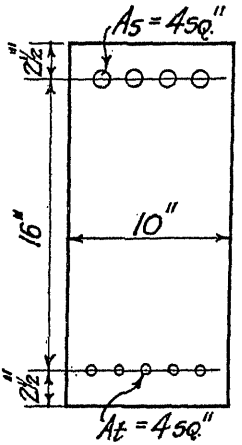


Fig. 54.

407. A compound girder over a shop front opening consists of two 12″ × 5″ × 30 lbs. I-beams with a 12″ × ½″ top flange plate. If the maximum shear force on a vertical section of the beam is 15 tons, analyse the shear stress in the web, give the maximum value, and sketch a diagram showing the distribution of horizontal shear stress throughout the depth of the girder. The sectional area of a 12″ × 5″ beam is 8·8 square inches, the web thickness is 0·33 inch and that of the flange, 0·5 inch. $I_{xx} = 207$ inches.[4] I. Struct. E. 1940.

408. A beam spanning between walls 20 feet apart, carries a load of 1 ton per foot run inclusive and is formed of an R.S.J. 16″ × 6″ × 62 lbs. Owing to erection difficulties it is found necessary to have the beam in two pieces, the joint being 7 ft. 6 in. from one wall face. Design and give a detailed sketch of the splice assuming rivets cannot be used.

I. Struct. E. 1940.

409. A light roof truss resting on 13½-inch brick walls has a span of 35 ft. 0 in. between centres of bearings and has a pitch of 30 degrees. The spacing of the trusses is 15 ft. 0 in. centre to centre and the roof covering is of asbestos cement sheets, steel purlins and glazing. Using electric arc welding for connecting the members, make a sketch showing the type of truss you would adopt, having due regard to economy, and make a detail sketch of the shoe of the truss. I. Struct. E. 1940.

410. A plate girder spanning 50 feet and composed of 20″ × 1½″ flange plates, 2/6″ × 3″ × ¾″ flange angles and 48″ × ½″ web plate was found to be damaged in the web, and the authorities have asked for it to be strengthened so that no stress need be carried by the original web plate. The flange plates and angles are quite sound. Design and make a sketch showing the necessary work required to be carried out.

I. Struct. E. 1940.

Detail drawings for both questions must be to a scale of not less than ¾ inch = 1 foot.

411. In Fig. 55 is shown a suggested section for a sports stand. The roof principals and back stanchions are to be spaced at 15 ft. 0 in. centres and the front main stanchions at 60 ft. 0 in. centres. Calculate in a practical manner the size of the members required and make a drawing to a scale of ⅜ inch to 1 foot showing the design of the roof, stanchions, any framing and the foundations. The seating to the stand is to be of

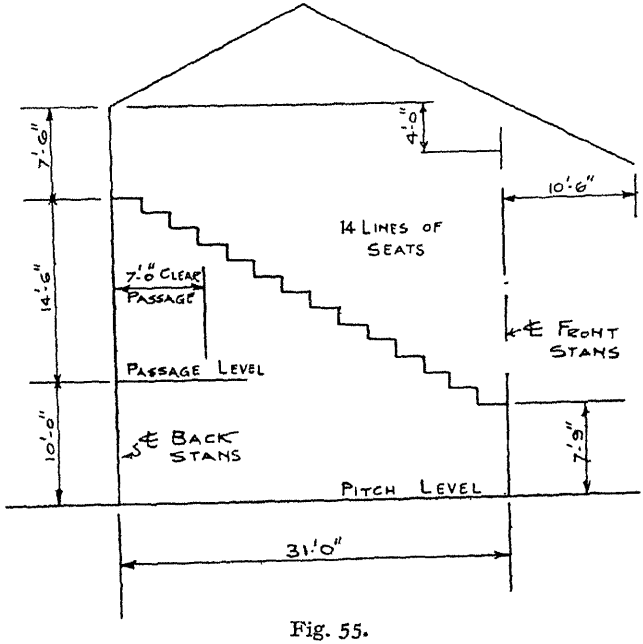


Fig. 55.

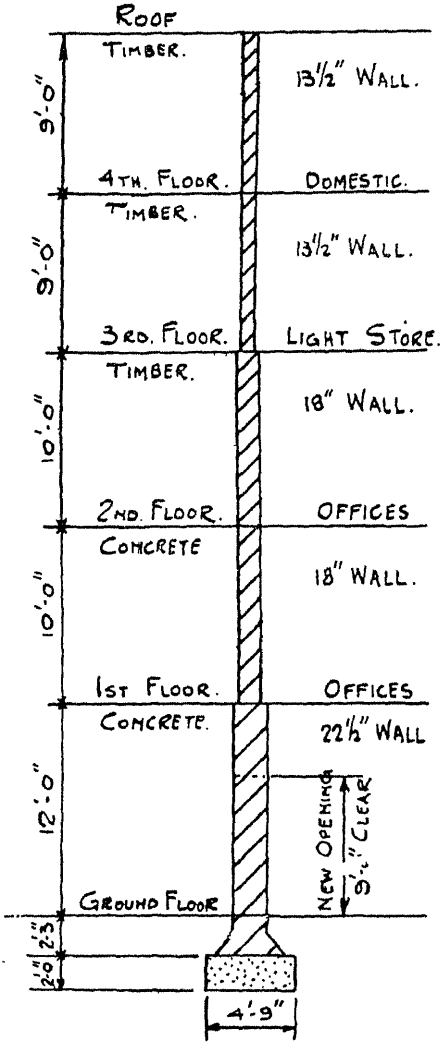

ROOF
TIMBER.
13½" WALL.
9'-0"
4TH. FLOOR.
DOMESTIC.
TIMBER.
13½" WALL.
9'-0"
3RD. FLOOR.
LIGHT STORE.
TIMBER.
18" WALL.
10'-0"
2ND. FLOOR.
OFFICES
CONCRETE
18" WALL.
10'-0"
1ST FLOOR.
OFFICES
CONCRETE.
22½" WALL
12'-0"
NEW OPENING
9'-2" CLEAR
GROUND FLOOR
2'-3"
2'-0"
4'-9"

Fig. 56.

timber construction and the side and roof coverings may be of any material you think suitable. The passage is to be formed of a concrete slab and the allowable pressure on the soil may be taken at 2 tons per square foot.

412. In Fig. 56 is shown a dividing wall between two buildings. The roof and floor construction is spanning on to this wall from beams 15 ft. 0 in. away each side. It is required to obtain direct access between these buildings and it is proposed to make a rectangular opening in the wall between the ground and first floors. This opening is to be 9 ft. 0 in. in the clear in height and 15 ft. 0 in. in width. Give calculations and make a drawing showing the sizes of members and foundations required for the necessary supporting frame and state the method of procedure you would adopt if you were supervising the operations.

I. Struct. E. 1940.

413. Fig. 57 shows a three-hinged arch rib ACB composed of two straight members AC and BC supported at A and B and meeting at the centre hinge C. AC = BC. There are equal loads of 5 tons applied normally at the centre of the lengths AC and BC.

Find : (*a*) The reactions at A and B.
(*b*) The force on the hinge at C.
(*c*) The section modulus required if the bending stress is 6 tons per square inch.

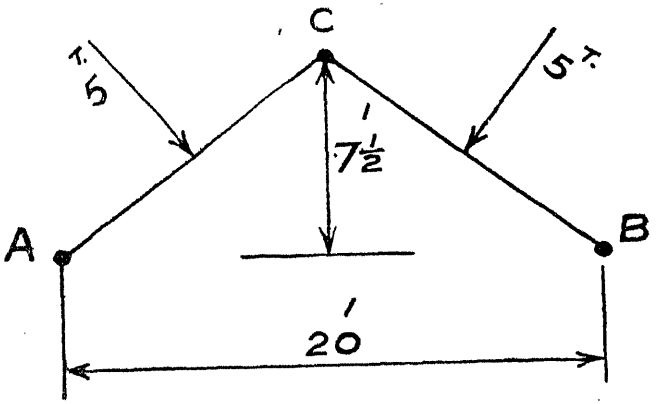


Fig. 57.

414. In testing specimens of timber explain how you would determine :—

(i) The stress at the proportional limit.
(ii) The modulus of rupture of the material.

What assumptions are made in arriving at the latter value and what are the uses of this modulus ?

I. Struct. E. 1942.

415. A braced double cantilever shown in Fig. 58 has panels 10 feet long by 10 feet deep. Find the forces in all the members due to the loading indicated and show clearly which members are in tension and which in compression. I. Struct. E. 1942.

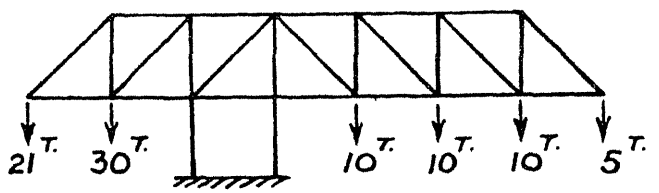


Fig. 58.

416. ABCD is a beam 9 feet long. AB = BC = CD = 3 ft. There is a uniformly distributed load from B to C of 6 tons per foot acting downward and supported by a uniformly distributed upward reaction from A to D of 2 tons per foot.

Draw the shearing force and bending moment diagrams giving all the important values and name one practical example of this state of loading. I. Struct. E. 1942.

417. A uniform continuous beam ABC is supported at A, B and C at the same level. AB = BC = 10 feet.

There is a 2-ton load 4 feet from A and a 3-ton load 5 feet from C.

Draw the shearing force and bending moment diagrams, giving all important values. I. Struct. E. 1942.

418. A mass concrete retaining wall is 3 feet thick at the top, 6 feet thick at the base and 15 feet high, with one face vertical. It supports a horizontal load of 2,000 lbs. per foot run acting on the vertical face at a height of 5 feet from the base. Find the distribution of vertical pressure on the ground if the concrete weighs 140 lbs. per cubic foot. What horizontal load would just reduce the pressure at one edge of the base to zero ? I. Struct. E. 1942.

419. A plate girder ABC, of constant section and 50 feet long, has two supports A and B, these being 22 feet apart. The girder overhangs 28 feet beyond support B, and is anchored at A. A point load of W tons is carried at the free end, C. (Fig. 59.)

If the maximum bending stress in the girder, due to this

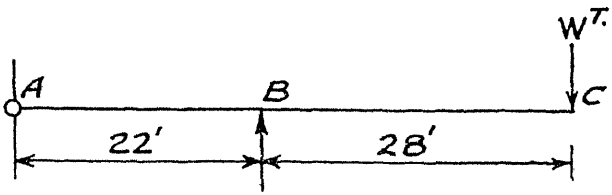


Fig. 59.

load, is to be 8 tons per square inch, and the deflection at C is to be 2 inches, what must be the minimum depth of the girder ?

If W is 5 tons, calculate the value of the moment of inertia of the girder section.

I. Struct. E. 1942.

420. A 12″ × 6″ at 44 lbs. B.S.B. stanchion is 30 feet high and is hinged at the top and fixed at the base. It carries an axial load of 6 tons. At a distance of 7 feet from the top, a flange bracket load of 10 tons is carried, the load acting 15 inches from the axis, and the bracket being 2 feet deep. (Fig. 60.)

Using any suitable method, find the moment of fixation at the base B.

Neglecting the stanchion's own weight, find the maximum compressive stress in the shaft, and state where it occurs.

I_{xx} = 316·76 inch^4 units. Area = 13·00 square inches. I. Struct. E. 1942.

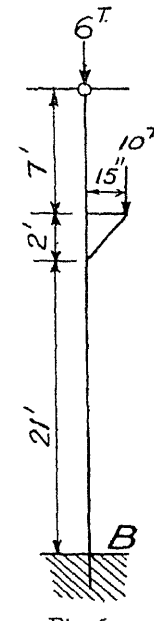


Fig. 60.

421. Obtain a formula which will give the intensity of shear stress at any point in a vertical cross section of a beam.

Use this expression to find the uniformly distributed load which can be carried by the mild steel beam shown in section in Fig. 61,

if the maximum shearing stress is limited to 6 tons per square inch.

Find the span, if the maximum bending stress is to be 8 tons per square inch. I. Struct. E. 1942.

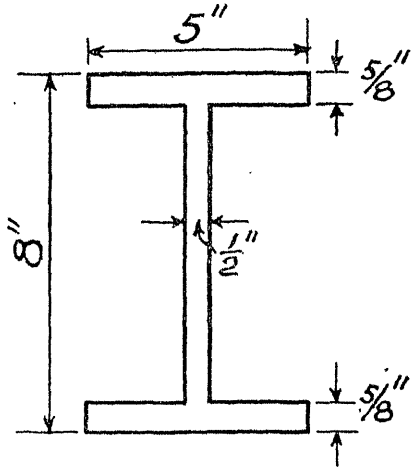


Fig. 61.

422. State and discuss the Principle of Least Work.

A rectangular portal frame ABCD (20-foot span ; 10 feet high) is hinged at A and D and carries a concentrated load of 10 tons on the horizontal beam BC at 4 feet from B. The moment of inertia of the section is constant throughout. By the Principle of Least Work, or otherwise, calculate the necessary values, and draw a dimensioned bending moment diagram for the frame. I. Struct. E. 1942.

423. Derive the Theorem of Three Moments for a beam whose moment of inertia is constant, and whose superimposed load is either irregular or uniformly distributed over both spans.

A continuous beam ABC covers two spans, AB and BC, both 20 feet long. The end A is firmly fixed in a horizontal direction, and the end C is freely supported. A, B and C are all at the same level, and the moment of inertia of the beam section is constant. There is no load on AB, but the span BC carries a concentrated load of 5 tons at 4 feet from B. Draw the bending moment diagram only. I. Struct. E. 1942.

424. Design and make a detailed drawing to a scale of $\frac{1}{2}$ inch = 1 foot of a gangway bridge suitable for spanning between a quay wall and a pontoon. The top of the pontoon is 2 feet above the top of the quay at high water, but provision must be made for a 12-foot fall of the tide. The gangway is to be suitable for carrying pedestrian traffic and light motor cars not exceeding 3 tons gross weight. The gradient of the deck of the gangway must not exceed 1 in 7. Show enlarged details of the bearings at both ends of the bridge.

I. Struct. E. 1942.

425. Design the steel framing for an engineering workshop to the following particulars :—

Length 150 feet, span 60 feet, height to underside of roof framing 20 feet.

A crane gantry for an electric overhead travelling crane of 3 tons lifting capacity is required, the crane spanning the full width of the building.

The roof and walls to be covered with galvanized corrugated iron sheeting, 20 gauge. A suitable amount of roof glazing and side windows are required.

Make an outline drawing to a scale of $\frac{1}{8}$ inch = 1 foot, clearly indicating the arrangement of the whole of the steelwork. Calculate the required sizes of the stanchions, gantry girders and roof truss members and make a detail drawing showing one-half of the cross-section of the building to a scale of $\frac{3}{4}$ inch = 1 foot.

The length of the stanchions may be shown curtailed.

I. Struct. E. 1942.

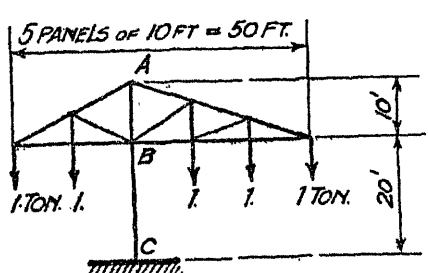


Fig. 62.

426. In the platform roof shown in Fig. 62 the column ABC is continuous and built into a foundation at C. The roof members are hinged to one another, and to the column at

A and B. Determine graphically or otherwise the forces in the members of the roof truss, indicating whether they are in tension or compression, and sketch the bending moment diagram for the column ABC. I. Struct. E. 1942.

427. A cast-iron I-beam is 12 inches deep overall and the web and flanges are 2 inches thick. The upper (compression) flange is 6 inches wide and the lower flange 12 inches wide. If the beam is subjected to a bending moment, find the ratio of tensile to compressive stress. What is the maximum load which the beam can carry at its centre, if it is simply supported on a 20-foot span ? The tensile stress must not exceed 2 tons/square inch, and the compressive stress 8 tons/square inch. Take the density of cast-iron to be 450 lbs./cubic foot

I. Struct. E. 1942.

428. A beam ABCDE is simply supported at A and D and carries loads of 2 tons and 1 ton respectively at B and C, and a load of 14 tons uniformly distributed over the whole length AE.

AB = 4 feet. BC = 8 feet. CD = 6 feet. DE = 10 feet. Sketch the shearing force and bending moment diagrams, giving all important values and the position of the points of maximum bending moment and contraflexure. Neglect the weight of the beam. I. Struct. E. 1942.

429. If two unequal loads, W and w, a fixed distance d apart roll across a simply supported span, show that the greatest bending moment occurs under the larger load W when the loads are in such a position that the centre of the span is midway between W and the centre of gravity of the two loads.

If the span is 36 feet, d = 10 feet, W = 6 tons, w = 4 tons. Find the maximum bending moment on the span and also the greatest values of the shearing force and bending moment at a point 6 feet from one end of the span. I. Struct. E. 1942.

430. A three-hinged arch ABC is formed of two straight members AB and BC supported at A and C, and meeting on the hinge B. AB = 13 feet, BC = 7·07 feet. The span AC is 17 feet and the rise from springings to crown is 5 feet. There is a uniformly distributed pressure of one ton per foot run of length on AB and BC acting normally to these members. Find the direction and magnitude of the force on each hinge and the maximum bending moment on AB.

I. Struct. E. 1942.

431. Show by sketches the distribution of (*a*) bending stress, (*b*) shear stress over the cross-section of a steel beam of I-section.

At a point on the cross-section of a rolled steel joist there is a tensile stress of 6 tons per square inch due to bending, and a shear stress of 2 tons per square inch. Calculate the magnitudes of the major principal stresses and the maximum shear stress at this point, and find their direction relative to the plane of the cross-section of the beam. I. Struct. E. 1942.

432. A 10″ × 8″ Tee section has a flange 8″ × 1½″, and a web 8½″ × 2″, both parts being rectangular.

Calculate (*a*) the moment of inertia of the section about an axis through the centroid and parallel to the flange.

(*b*) Calculate the least radius of gyration of the section.

I. Struct. E. 1942.

433. A concrete beam, 10 inches wide, is reinforced on the tension side with material having a safe tensile stress of 10,000 lbs./square inch and a modulus of elasticity of 10×10^6 lbs./square inch, the material being centred at a distance 16·5 inches from the compression face.

If "A" is the cross sectional area of the material, and "M" the moment of resistance of the reinforced beam, draw a graph showing the relationship between M and A.

I. Struct. E. 1942.

434. The side columns of an existing building are of 8″ × 4″ B.S. beam section having an overall length of 19 feet and are spaced at 12 feet 6 inch centres. Each column supports a concentric load of 4 tons from the shoe of a roof truss. The trusses, which are comparatively very stiff and are effectively fixed at their other ends, may be assumed to support or "prop" the heads of the columns.

The columns are only tied together by sheeting angles connected to their outer flanges, the rails being at 1 foot, 7 feet, 13 feet and 19 feet above ground level. The side of the building is covered with galvanized corrugated iron. The column bases are at ground level, and are effectively anchored to concrete foundations.

The building is in an exposed position near the sea. Make an allowance of 15 lbs. per square foot for wind pressure and calculate the stresses in the column. State whether these stresses are considered to be excessive and, if so, what safety measures should be adopted.

The following are the properties of the section of the column: Area = 5·3 square inches. Maximum Moment of Inertia = 55·63. Minimum Moment of Inertia = 3·51.

I. Struct. E. 1942.

435. Design and make a detailed sketch of a concrete foundation and steel base with anchor bolts for a 12″ × 6″ rolled steel joist column subject to the following conditions of loading :—

(*a*) Maximum vertical load on column = 30 tons.
(*b*) Minimum ,, ,, ,, ,, = 10 tons.
(*c*) A horizontal load of 1 ton in the direction of the web of the column applied at 12 feet above the top of the concrete foundation.

The horizontal load may occur with either the maximum or minimum vertical load. The foundation is in compact gravel and the maximum load on this material may be taken as 4 tons per square foot. I. Struct. E. 1942.

436. A steel bracket is 18 inches deep and is bolted to the flange of a rolled steel joist column with two rows of $\frac{7}{8}$-inch diameter black bolts. The pairs of bolts are $2\frac{1}{4}$ inches, $6\frac{3}{4}$ inches, $11\frac{1}{4}$ inches and $15\frac{3}{4}$ inches above the bottom of the bracket.

The cross sectional area at the bottom of the thread of a $\frac{7}{8}$-inch diameter bolt is 0·42 of a square inch.

The bolts are screwed up so tight before any load is applied to the bracket that a tensional stress of 1 ton per square inch of net sectional area is induced in the shanks of the bolts.

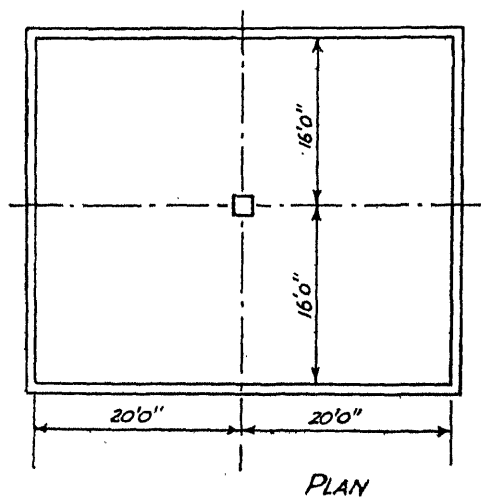


Fig. 63.

Calculate the maximum load that may safely be applied on the top of the bracket at 1 foot from the face of the column. Assume that the bracket is comparatively much stronger than the bolts which connect it to the column.

I. Struct. E. 1942.

437. Fig. 63 shows the rough plan of a proposed reinforced concrete roof over an underground room 32 feet by 40 feet between existing mass concrete walls 9 inches thick at the top.

The roof is to be designed to carry a live load of 300 lbs. per square foot.

A central column is to be provided carrying two lines of beams parallel to the walls; its base is to be 13 feet below the top of the roof slabs and 3 feet below the basement slab. The allowable ground pressure is 2 tons per square foot.

The candidate is required to give a complete design of the roof and central columns, and to take out quantities of concrete, steel and formwork. I. Struct. E. 1942.

Steel in tension	... ...	8 tons per square inch.
Concrete in compression	...	750 lbs. per square inch.
Concrete in diagonal tension		75 lbs. per square inch.

438. A long railway station platform is 60 feet wide, and has a track running along each side of it.

Design the steelwork for a suitable platform roof, make a general drawing of a portion of the length of the roof to a scale of $\frac{1}{8}$ inch = 1 foot and a detailed cross section of one-half of one of the main frames to a scale of $\frac{3}{4}$ inch = 1 foot.

I. Struct. E. 1942.

439. Design and make a detailed drawing to a scale of $\frac{1}{2}$ inch = 1 foot of one-half of a steel lattice-girder supporting a through type highway bridge. The span of the bridge is 90 feet between abutments and has a carriage-way 30 feet wide with pedestrian paths each side 7 feet 6 inches wide. The loading is to be as required for heavy traffic.

I. Struct. E. 1942.

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